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Pecos River Cantilever

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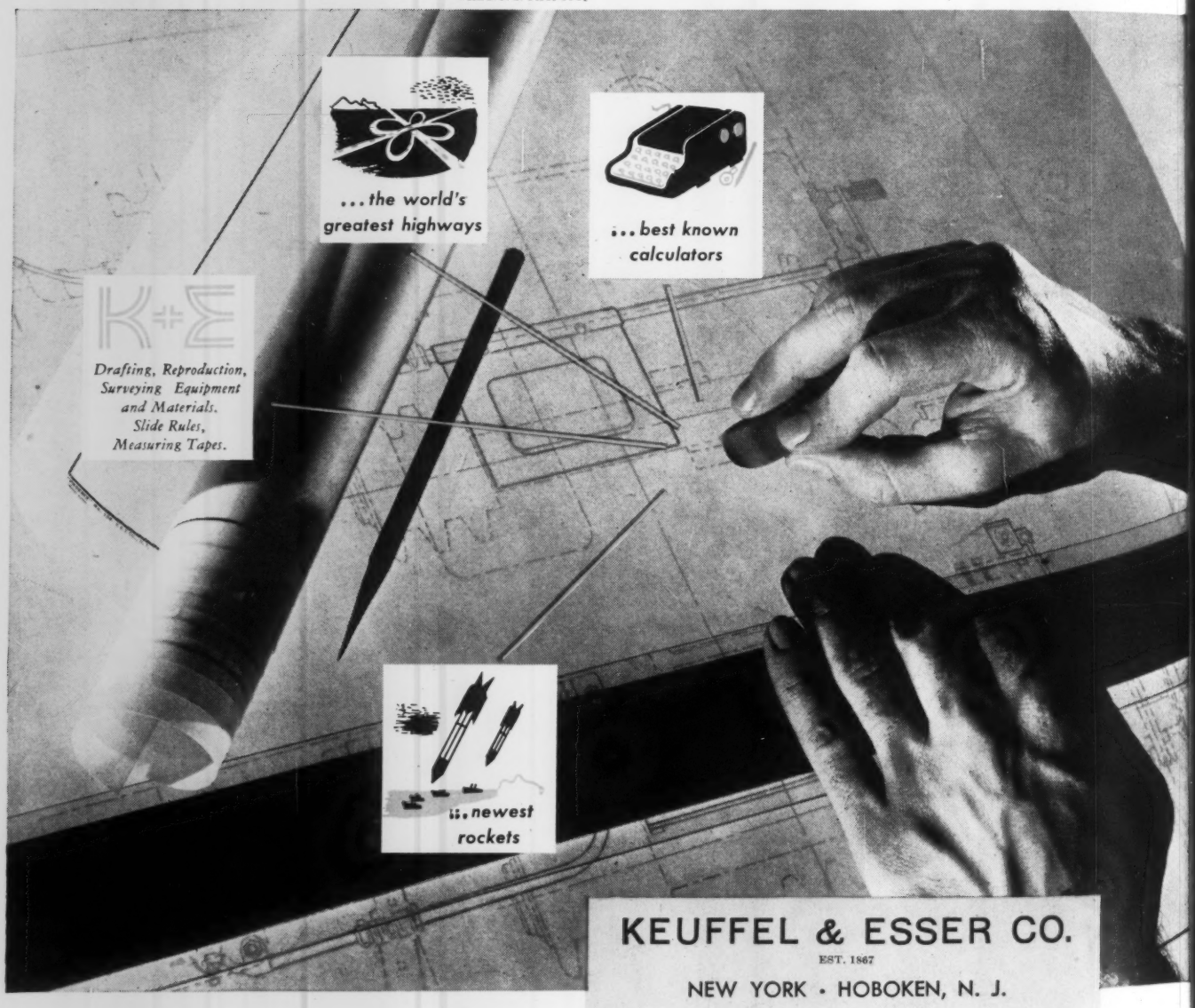
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G. H. HICKOX (U. of Iowa, B.E. '25, M.S. '26; U. of Calif., Ph.D. '39) worked for 3 years with the West Penn System and Morgan Engineering Co., later taught hydraulics at the U. of Calif. Since 1935 he has been in charge of TVA's hydraulic laboratory, where all model tests are made on the Authority's hydraulic structures.

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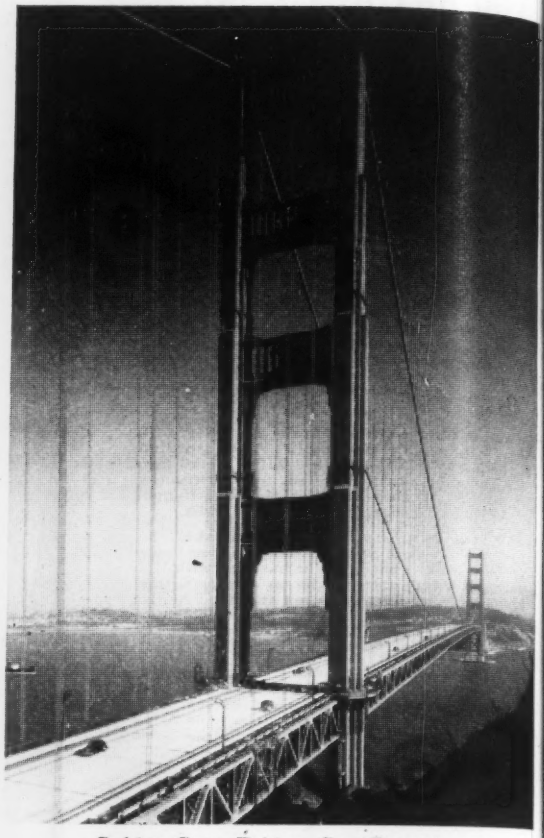

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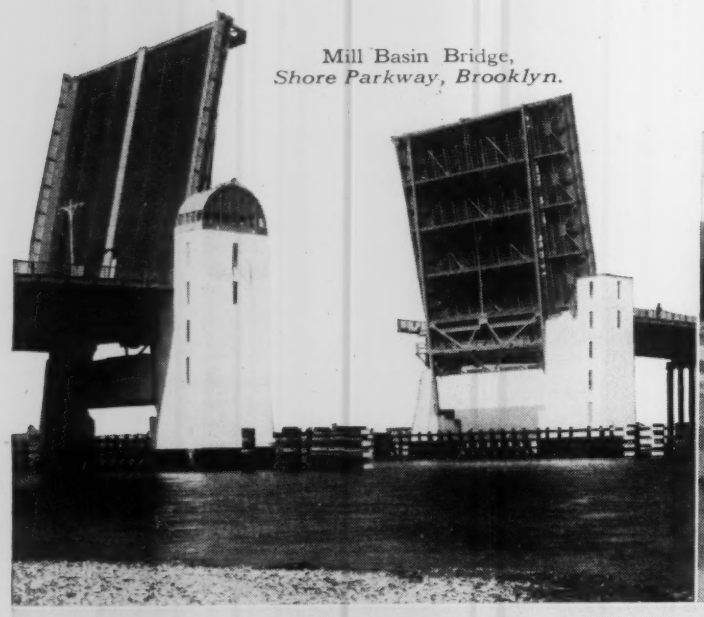

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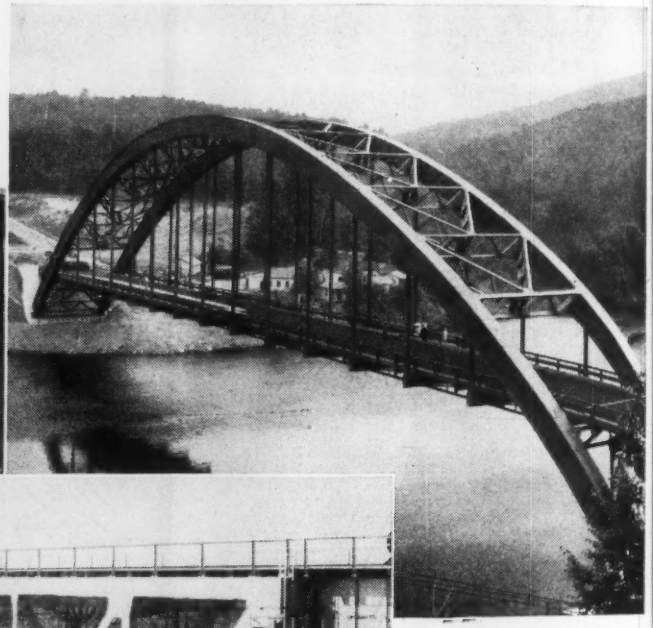

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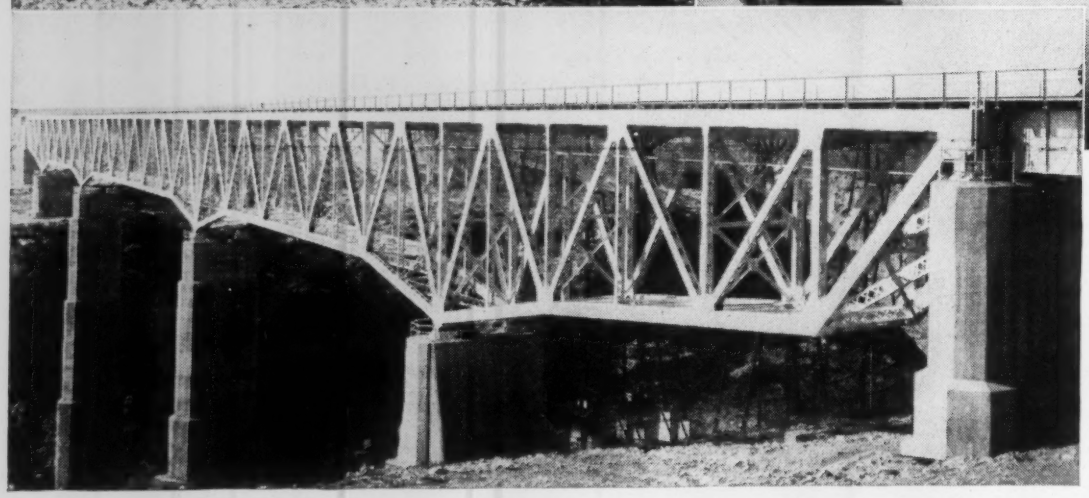

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Pecos River, High Bridge, Texas.

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Volume 15 — October 1945 — Number 10

More Water for San Diego

Aqueduct Under Construction to Deliver Additional 50 mgd from Colorado River

By Fred D. Pyle, M. Am. Soc. C.E.
Hydraulic Engineer, City of San Diego, Calif.

War years strained to capacity the water system of San Diego. Over 150% of the safe yield of the city's watersheds was used. Faced with a continuing demand in excess of available supplies, the city turned to the Colorado River resources to which it had maintained rights since 1926. The San Diego Aqueduct, now under construction, will tap the San Jacinto Tunnel of the Metropolitan Water District at its western end. Construction will be under the supervision of the Navy's Bureau of Yards and Docks.

LONG before V-J day, an increase in the water supply for San Diego, Calif., was not considered a postwar problem but an immediate one calling for: (1) construction by the Government within two years of the San Diego Aqueduct (capacity 50 mgd, length 71 miles, estimated cost $17,500,000) to make Colorado River water available so as to avoid possibly disastrous results in the event of droughts such as are common to the area; and (2) construction by the city of additional pipe-line capacity from the city's reservoirs on the San Diego River watershed to the distribution system, involving about 21 miles of 48- to 72-in. pipe, a modern 60-mgd water treatment plant, and a 20-million-gal treated-water regulating reservoir, estimated to cost $6,000,000.

In 1940 the City of San Diego had a population of 203,341 and about 20,000 additional in military service. The average use of water for the calendar year was 23.7 mgd, the average for the maximum month 31.0 mgd, and the use on the maximum day, 35.8 million gal. At that time the city had six impounding reservoirs and two large regulating reservoirs with a capacity of 105.8 billion gal and a safe yield of 26.6 mgd.

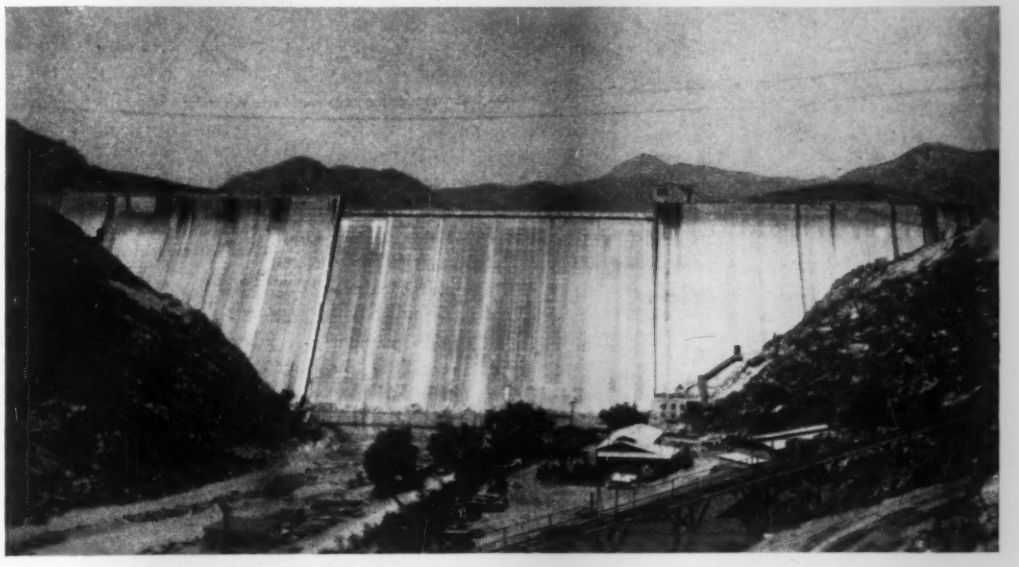
San Vicente Impounding Reservoir

Since 1940, the city has expended about $3,275,000 in constructing the San Vicente impounding reservoir (Fig. 1), with a capacity of 29.4 billion gal, and $1,525,000 on distribution mains and pumping plants; and the Federal Works Agency has expended about $2,200,000 on transmission mains, distribution mains, and a 10-million-gal regulating treated-water reservoir. San Vicente Reservoir will increase the safe yield of the city from local sources by 5.3 mgd when 75% filled. Local sources are now about 60% developed. Table I indicates the changes in population and water use during the past six years. The average use for July 1945 was 59.8 mgd and the maximum day, 69.2 million gal, of which the government and war agencies used 44.4%, or 30.7 mgd.

The following Navy institutions are of a permanent character and will continue after the war to use large quantities of water: Naval Air Station, Naval Supply Depot, Naval Hospital, Destroyer Base, Marine Base, and Naval Training Station.

Water deliveries are 100% metered; the soil is tight so that leaks come to the surface, and during the last half of 1944 the unaccounted-for water losses between impounding reservoirs and the city amounted to only 2.93%. These facts indicate that the safe yield of the city's reservoir system is being greatly overdrawn.

From November 1, 1936, to October 31, 1943, the average annual runoff of the city's watersheds was 158.1% of the 60-year mean from 1883 to 1943. During



San Vicente Dam, Newest Reservoir of the San Diego Water Supply System

SECTION OF TRENCH FOR SAN VICENTE PIPE LINE ACROSS SAN DIEGO RIVER
Excavated After Wellpoints Had Lowered Ground-Water Level

the period 1883 to 1943, there was extreme drought from 1897 to 1904, when the average annual runoff was only 7.5% of the 60-year mean. During a number of three-year periods there was but little runoff. During the 60-year period, only 16 years exceeded the 60-year mean, while 30 years were below 50% of this mean. The runoff is so erratic that State Bulletin No. 48, "San Diego County Investigations 1935," indicates that on the average about eight years are required after construction of an impounding reservoir before it can be depended upon for the theoretical safe yield.

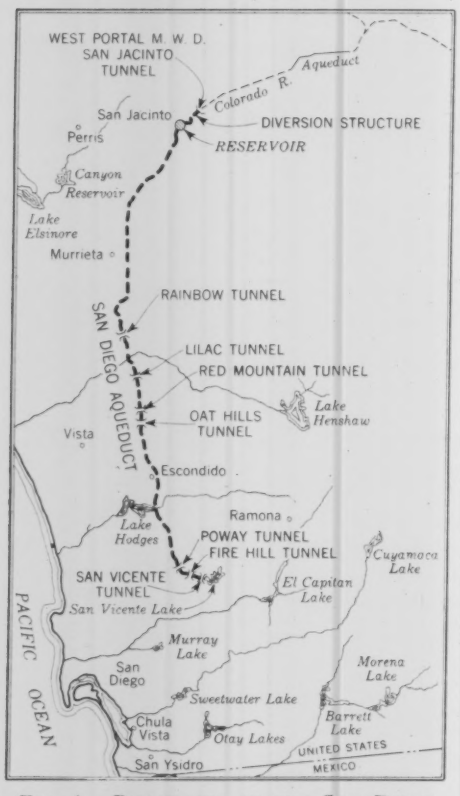


FIG. 1. RESOURCES OF THE SAN DIEGO WATER DEPARTMENT

Aqueduct from San Vicente Reservoir to San Jacinto Tunnel Under Construction

San Vicente Reservoir, completed in time to impound 95% of the 1942–1943 runoff, was on July 1, 1945, and with no water withdrawn, only 14.0% filled. For the period 1897 to 1904, the total runoff at the San Vicente dam site, without allowance for evaporation, amounted to only 4.3% of the reservoir capacity. Also on July 1, 1945, the city's reservoirs were 66.5% full. With estimated requirements and existing transmission mains, plus a booster pumping plant to increase the delivery of water from the Morena-Barrett-Otay system from 17.5 to 25.5 mgd commencing August 1, it is anticipated that should drought conditions prevail, the storage in the city's major reservoirs (El Capitan and San Vicente) and the Murray regulating reservoir, would be reduced to 10,000 acre-ft by July 1, 1947. At that time there would be about 60,000 acre-ft of water in the Morena-Barrett-Otay reservoir system.

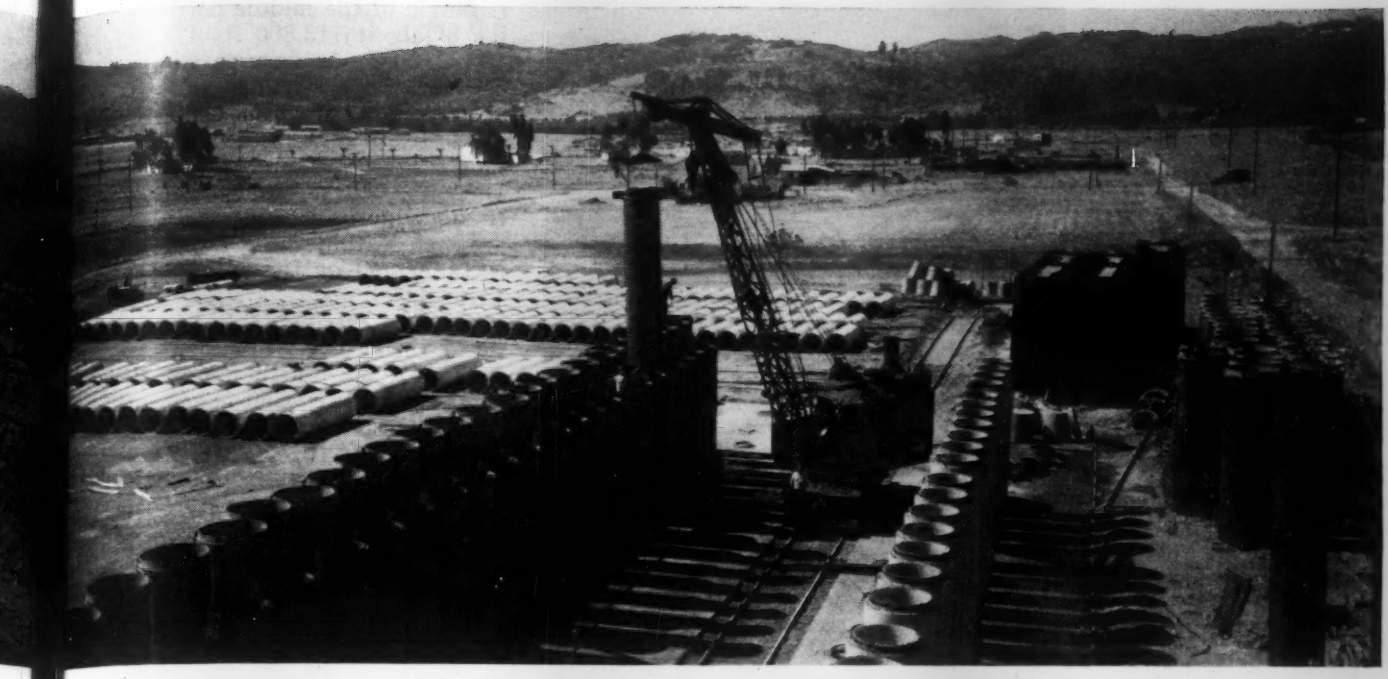
Anticipating the time when the full development of local water resources would be insufficient to meet its requirements, the City of San Diego in 1926 filed on 100 mgd of water from the Colorado River and since that time has diligently maintained its right by entering into various contracts with the U.S. Department of the Interior and the State Division of Water Resources.

TABLE I. CHANGES IN POPULATION AND WATER USE, 1939–1944

YEAR	POPULATION, CIVIL	POPULATION, MILITARY	AVG. USE Mgd	MAX. DAY Million Gal	% USED BY Gov't*
1939	197,000	17,500+	20.82	32.50	10.0
1940	205,000	20,000+	23.70	35.77	10.9
1941	265,000	50,000+	26.21	42.06	17.5
1942	315,000	100,000+	36.24	50.70	28.4
1943	340,000	100,000+	42.19	57.71	40.3
1944	360,000	100,000+	45.57	61.16	43.0

* Including military, United States housing, and national defense plants

The great increase in the use of water due to natural growth and war agencies made it evident that, to ensure a dependable supply, Colorado River water should be brought to the San Diego area. In May 1943 a contract was entered into between the Federal Works Agency, the Bureau of Reclamation, and the City of San Diego for a survey by the Bureau, to include engineering studies and an estimate of cost for a conduit from the west end of the San Jacinto Tunnel of the Metropolitan Water District of Southern California to the San Diego area. About the same time, another contract was entered into between the Bureau of Reclamation, the County of San Diego, and the City of San Diego, for a second survey by the



REINFORCED CONCRETE PIPE BEING MADE FOR SAN DIEGO DISTRIBUTION SYSTEM
El Cajon Plant of American Pipe and Construction Company

Bureau, embracing engineering studies and estimates of cost for a conduit to bring water from the All-American Canal to the San Diego area. The Bureau was also to make a comparison of the costs of the two conduit routes.

The San Diego County Water Authority was formed as a result of an election on May 16, 1944, with the following agencies as members: Cities of San Diego, Chula Vista, National City, Coronado, and Oceanside; Fallbrook Utility District, La Mesa Irrigation District, Ramona Irrigation District, and Lakeside Irrigation District. The Authority was formed to arrange for the delivery of Colorado River water for the use of the member agencies. The City of San Diego represents 82% of the assessed valuation of the Authority.

INTERDEPARTMENTAL COMMITTEE APPOINTED

Factors affecting the water supply of the City of San Diego, and the war agencies in and near it, were brought to the attention of the late President Franklin D. Roosevelt in the fall of 1944 by city and Naval officials. As a result he appointed, on October 3, 1944, a special interdepartmental committee to consider the problems and make recommendations. The committee consisted of William E. Warne, Assistant Commissioner, Bureau of Reclamation, U.S. Department of the Interior, Chairman; Vice Admiral Ben Moreell, Chief of the Bureau of Yards and Docks, Navy Department; Maj. Gen. Eugene Reybold, Chief of Engineers, War Department; Baird Snyder, Assistant Administrator, Federal Works Agency; and Phil D. Swing, Attorney, San Diego County Water Authority.

After considering the relative merits of the two routes, All-American and Metropolitan, on the basis of meeting emergency requirements for speedy construction and minimum use of critical materials and manpower, the committee recommended the immediate construction of the San Diego Aqueduct from the west end of the San Jacinto Tunnel of the Metropolitan Water District of Southern California to San Diego's San Vicente Reservoir (as shown in Fig. 1), the pipe lines to have a capacity of 50 mgd and the tunnels that of 100 mgd (if costs were favorable when compared with a 50-mgd capacity).

As a result of the study and recommendations, in November 1944 President Roosevelt directed the Bureau of Reclamation to complete the plans and specifications with the cooperation of the Army, Navy, and Federal Works Agency; and directed the Navy to construct the aqueduct with the cooperation of the other three agencies. The President requested the City of San Diego and the San Diego County Water Authority to negotiate with the Metropolitan Water District of Southern California for delivery of water at the San Diego Aqueduct intake.

On December 18, 1944, at a conference held in the office of the Public Works Officer in San Diego, attended by 18 representatives of the interested agencies, a program of procedure was outlined. The Bureau of Reclamation proceeded immediately with the detailed field location and land ties, and the preparation of designs, drawings, and specifications.

The Bureau has completed the field work and designs and has delivered the drawings and specifications to the Navy. The principal items include a 1,400-acre-ft regulating reservoir 2 miles from the upper end of the aqueduct; 7 tunnels (6 by 6 ft, horseshoe shaped, concrete lined) with a total length of 4.3 miles; 1.8 miles of high-head 48-in. plate-steel pipe across river valleys; and about 2.0 miles of 96-in., 12.2 miles of 72-in., 21.9 miles of 54-in., and 28.2 miles of 48-in. concrete pipe with various types of reinforcing, depending upon the pressure head. As designed, the water surface at the intake of the aqueduct will be at El. 1,504.7, and at the delivery point to the San Vicente Reservoir, at El. 760.0.

A contract has been let to the American Pipe and Construction Company for joint rings and gaskets for the entire project, at an estimated cost on the basis of the contract, of $563,282.50. Nine informal bids were received by the Navy on April 4, 1945, for the Poway, Fire Hill, and San Vicente tunnels, which are near the lower end of the aqueduct. Award has been made to the W. E. Callahan Construction Company and the Gunther and Shirley Company, low bidders, for the construction

ONE OF THE NEW LINES TO BRING WATER INTO THE CITY
Reinforced Concrete Pressure Pipe Being Laid

of these three tunnels (total length 11,280 ft) at a contract price of $868,536, which includes the furnishing of all materials.

Eight informal bids were received by the Navy, on July 6, 1945, for 48,040 ft of 72-in. pipe, 41,540 ft of 54-in. pipe, and 10,900 ft of 48-in. pipe. The 72-in. pipe is gravity flow, and the remainder low pressure, none exceeding 125-ft head. A large portion of the gravity-flow pipe is in 10 to 20-ft cuts. The plans call for reinforced poured concrete pipe with lock joints. There are but few incidental structures, mostly access manholes. The low bid of $2,248,409, which includes all excavation, backfill, pipe, and materials (except joint rings and gaskets), was submitted by the W. E. Callahan Construction Company. This section of the aqueduct extends in a southerly direction from the proposed reservoir. Contract has been awarded to the low bidder.

Seven informal bids were received by the Navy on August 1, 1945, for the construction of 114,500 ft of aqueduct, consisting of 2,225 ft of 72-in. pipe, and 112,275 ft of 48-in. pipe with appurtenant structures consisting of blow-off valves, air valves, manholes, access structures, and bifurcation structures. The plans call for reinforced poured concrete pipe with lock joints, and for reinforced steel-cylinder concrete pipe with lock joints. This is for the lower part of the aqueduct. Contract has been awarded to the low bidder, S. A. Healey, on the basis of a bid of $2,597, 344.70.

Six informal bids were received by the Navy on August 8, 1945, for the construction of Rainbow, Lilac, Red Mountain, and Oat Hill tunnels with a total length of about 11,750 ft. Contract has been awarded to the low bidder, S. A. Healey, on the basis of a bid of $264,080.

Twelve informal bids were received by the Navy on August 22, 1945, for the construction of the aqueduct from the intake to and including the Regulating Reservoir, consisting of 10,430 ft of 96-in. low-head concrete pipe and 874,000 cu yd of excavation, backfill, and embankment, also appurtenant connections, control works, and meters. Contract has been awarded the low bidder, the Guy F. Atkinson Company, on the basis of a bid of $1,164,885.

As for the rest of the aqueduct, informal bids will be received by the Navy on September 28, 1945, for the construction of the middle portion consisting of about 113,800 ft of 48, 54, and 72-in. concrete pipe with various types of reinforcement. Soon the Navy expects to issue a call for bids for about 9,560 ft of high-pressure 48-in. steel pipe across the San Luis Rey and San Dieguito valleys, which will complete the aqueduct. The bids thus far received are well within the estimated cost of the aqueduct on the basis of a total of $17,500,000.

The War Production Board has stated that no steel sheet or plate will be available until the fourth quarter of 1945, and has requested that no miners be employed on the tunnel work until after November 1, 1945. This has caused some delay, but it is believed that the aqueduct can be completed in time to commence delivery of Colorado River water in the summer of 1947. Excavation for the aqueduct was commenced by the Guy F. Atkinson Company near Hemet on September 12, 1945.

The electorate of the City of San Diego, on April 19, 1945, with a 10-to-1 vote, authorized a $6,000,000 bond issue for additional pipe-line capacity from the San Vicente reservoir to the city; also a modern 60-mgd water treatment plant, and a 20-million-gal regulating treated-water reservoir. These facilities will be used for local water when available, and for Colorado River water or a mixture of the two sources when required.

It is anticipated that the pipe lines and regulating reservoirs will be completed in time for use in the 1947 summer season, and that the treatment plant will be completed by the late fall of 1947.

Completion of the San Diego Aqueduct, additional mains and treatment plant, will assure the Navy, the City of San Diego, and the other agencies of the San Diego County Water Authority, a reliable water supply and make possible the orderly conservation by the city of additional local water supply, including construction of the Pamo and Sutherland reservoirs and conduits to the San Vicente Reservoir, an increase in the capacity Hodges Reservoir by construction of a new dam below Hodges Dam, and the heightening of Barrett Dam, with the necessary conduits to the city.

Construction work on the San Diego Aqueduct is being done under the direction of Capt. Alden K. Fogg, Public Works Officer. Comdr. R. C. Thorson, CEC, U.S.N.R., is Resident Officer in Charge.

Work by the Bureau of Reclamation is under the direction of H. W. Bashore, director; designs and specifications are under the direction of Walker R. Young, chief engineer, and E. A. Moritz, regional director. The engineer in charge of the Bureau's work on the aqueduct is R. B. Ward.

J. L. Burkholder is chief engineer and general manager of the San Diego County Water Authority.

Carl R. Rankin is consulting engineer for the city on the location and design of pipe lines and reservoirs; J. M. Montgomery is consulting engineer on design of the treatment plant. Paul Beermann is engineer in general charge of surveys, design, and construction. W. C. Brown is associate engineer in charge of office engineering and specifications. The writer is hydraulic engineer, in charge of water development, transmission, and purification. All those mentioned are members of the Society.

Railway Roadbeds Stabilized with Portland Cement Grout

By ARTHUR J. BOASE, M. AM. SOC. C.E.

MANAGER, STRUCTURAL BUREAU, PORTLAND CEMENT ASSOCIATION, CHICAGO, ILL.

PRESSURE grouting has long been used to stabilize foundation soils, but its application to water pockets under railway track and to "soft track" in general is a more recent development. The first use of pressure grouting to stabilize railroad track was on the Pennsylvania Railroad at North Point, Md., in 1936. This and other early installations have now been in long enough to prove definitely that pressure grouting with portland cement grout is not a temporary palliative but offers a long-time cure for troublesome track. The greatly reduced cost of track maintenance, and the release of desperately needed labor for other railroad work after track has been grouted, have made grouting a practical economy.

UNDER the high pressures developed by passing trains, certain types of subgrade will yield, and in some places local depressions, known as water pockets, are formed. These are of irregular cross section and length; some extend only a few feet, others for a mile or more. They may occur in cuts of any depth, on fills of any height, but particularly in soils through which water does not readily drain. When manipulated under pressure, these soils soften, become almost liquid, and are easily eroded by moving water. The pulsating action of the passing trains causes this liquefied material to move, ejecting it through vents and porous areas, coating the ballast, clogging French drains, and choking drain tile, catch basins, and other drainage structures. The use of drains and other methods of stabilization, such as closely spaced poles, is costly and has often proved ineffective.

A typical cross section of a water pocket (Fig. 1) reveals a relatively impervious top layer of varying thickness in which voids in the ballast have been filled with fine cinders, engine sand, and pulverized material, and often impregnated with liquefied clay forced in from below. The ballast material below this top layer is usually quite porous, being kept open by the surging of the water under pressure. This porous section extends downward nearly to the clay line. Liquefaction commences at the clay line, and the fine material is deposited on the top, sides, and bottom of the pocket, gradually sealing these surfaces. Internal hydraulic pressures are produced which cause the roadbed to yield, resulting in bad-riding and sometimes dangerous track. In cuts the material is forced outward from under the track and then upward. On fills, bulges may occur opposite the pocket near the top of the shoulder or on the side slopes considerably below the shoulder. The soft material is sometimes forced up to the surface between tracks and between rails, sometimes to the extent that the ballast is permeated with a viscous mud.

Pressure grouting has not been confined to water pockets but has also proved beneficial in most places where there is a soft track condition. Maintenance on stretches of soft track, or where there are water pockets, is extremely high, as such track must often be gone over three or four times a week, and ballast must be frequently removed. On some of the areas treated by grouting the cost of the work has been little more than a month's maintenance cost, and in most locations it has been equivalent to only a few months' maintenance cost. The subsequent savings in maintenance have paid the cost of the grouting many times over in a period of a few years.

The grouting of these areas with portland cement grout injected under pressure seals off the water-retaining depressions, thereby preventing moisture from reaching the unstable subsoil. Grout deposits form barriers which prevent surging action of the liquefied material, and concrete formations are produced which help distribute the traffic load over the soft depressions. The original load-bearing capacity of the subgrade is thereby restored and even increased. Even when actual concrete slabs are not formed, there is an action between the cement and the subgrade material which results in a stabilization of the track.

Much of the grouting work to date has been done with equipment and accessories improvised from material and parts usually found in railroad shops. Some of the roads are, however, purchasing and using combined mixing and grouting machines that have been developed to meet this demand. The injection of the grout can be done with either pneumatic or hydraulic pressure, but pneumatic equipment has been most generally used.

Exponents of the pneumatic method contend that the air tends to open up passages for the grout, thereby securing better penetration. The air is usually supplied from a compressor of the type and capacity furnished for tamping ballast with air tools. Pneumatic equipment is limited, however, to pressures not much in excess of 100 lb per sq in., and in fact it is found that pressures of about 40 lb per sq in. frequently result in better penetration than higher pressures. In exceptional cases, where voids and passages in the material to be grouted are small, or where water must be displaced from fine-textured materials, pres-

GROUTING TRACK ON THE WABASH NEAR GRABILL, IND., WITHOUT INTERFERENCE WITH TRAFFIC

Injection Points Have Been Driven and Grouting Is Being Done with Horizontal Pneumatic Pressure Tank

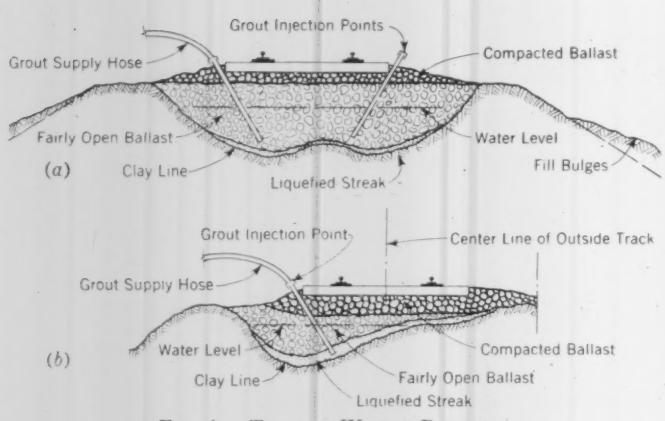


FIG. 1. TYPICAL WATER POCKETS

(*a*) Under Single Track on a Fill—Grout Is Usually Injected from Both Sides of Such Track; (*b*) Under Outer Rails of Double Track—Generally Remedied by Injection Points Along Outer Rails Only

sures from 150 to 300 lb per sq in. may be necessary. Hydraulic pressure is ideal for such conditions as it is built up gradually until the grout is accepted, and when flow commences there is no disruptive violence. A single-acting plunger pump is usually used on such equipment.

When equipment is set up at the side of the track, grouting can proceed without interruption to traffic. The grout discharge line usually consists of a short length of pipe from the pressure tank and then a flexible rubber hose, such as a steam hose. This is coupled to an iron pipe injection point, which has been driven to the proper depth for grouting. Where high resistance to driving of the injection points is encountered, a solid steel spud bar is first driven to provide a hole for the injection point. The varying under-track conditions can be sensed accurately by the resistance encountered when points are driven. When there is doubt as to the ability of a pocket to take grout, a few widely spaced points are driven for test by injecting water under pressure. If water is not received, grout would not be accepted at this point. Injection points are usually spaced at about every second crib, driven somewhat outside the ends of the ties but at an angle with the vertical so that the tip is approximately under the rail when it is in grouting position. Work starts at one end of the pocket or soft area and moves progressively forward.

The proportions of cement, sand, and water will depend on the character of the sand and the condition of the subsoil. Excess water in the grout may result in separating water and cement from the mix when under pressure. The proportioning is usually done by trial, usually starting with a very fluid, neat cement grout. If this is readily injected, fine sand is added in increments until the desired consistency is obtained. Some engineers limit the sand to 200 lb per sack of cement, and if this gives too fluid a grout, the amount of water is reduced. When the sand is extremely fine and contains some silt and clay, somewhat more of it can be used.

Up to the beginning of 1945, stabilization of roadbed by cement grouting on the New York Central had been confined to its lines west of Buffalo, but this year work will also be done on its eastern lines. Up to January 1, 1945, 197 soft spots had been treated. These involved a total of 36,379 lin ft of track. Carefully kept records show a sharp reduction in maintenance cost in every instance. In 1942 a total of 4,907 ft of track at 31 spots was treated at a total cost of $4,334.11. Maintenance had cost $1,187.10 a month before treatment and has been only $73.95 a month since treatment, or a saving of $1,113.15 a month. The entire cost of grouting this track was paid for by the saving in maintenance in less than 4 months. Furthermore, this saving has continued to date. In 1944, costs were higher. In that year 84 locations involving 17,943 ft of track were treated at a cost of $32,481.61. The indicated savings in maintenance on that year's work are $3,691.19 a month.

Results at individual spots are even more startling. For example, two spots which required maintenance not less than three times a week were costing $122.64 and $126.32 a month. Grouting was done at costs of $165.12 and $135.76, with subsequent reductions in maintenance costs to $3.10 and $14.60 a month, respectively. As a result, the grouting of these two holes not only paid for itself but saved the company $2,474.24 the first year. Some soft spots which formerly needed frequent maintenance have required no maintenance whatever in the two years or more since they were grouted.

The Santa Fe has done more pressure grouting than any other road to date. The first installation was in 1941, with additional stretches in 1942 and 1943, so that by the end of 1943 about 30 miles of track had been treated. In 1944, a major program of grouting the heavy-duty track between Holliday and Emporia, Kans., was adopted. The work covers a 100-mile stretch of track which constantly had been troublesome, having deep cuts, high fills, curves and grades, in soil that is predominantly soapstone and clay. Carefully kept records showed abnormal maintenance and a history of frequent slow orders for this track. Several methods had been tried to remove water from the pockets, including the installation of drains. These worked well for a time, but gradually filled up and became ineffective.

Ten grouting outfits—three of them hydraulic machines developed by the Santa Fe, and the other seven commercial hydraulic units—worked simultaneously on this project. Availability of an extremely fine sand, called blow sand, nearly all of it passing a 50-mesh sieve, has permitted leaner mixes than are ordinarily used. Experience shows, however, that the cement cost is less than 20% of the cost of the work, indicating that the amount of cement used does not greatly affect the overall cost, and that any appreciable reduction in cost can be realized only by reducing the cost of injection.

All treated track has shown improvement both in appearance and in riding performance and has revealed a marked saving in labor for track maintenance, ranging from 30 to 82%. At most locations the track has continued to improve after grouting, and

VERTICAL PRESSURE TANK USED ON THE BALTIMORE AND OHIO FOR GROUTING AT THE INTERLOCKING PLANT AT NILES JUNCTION, OHIO

Note Hose Line Under Track So as Not to Interfere with Traffic

maintenance has become progressively less. In many locations a more important result is the increase in train speed permitted. An example of this is the Thatcher line change between Delhi and Simpson, Colo. When heavy rains occurred at this point, speed restrictions to as low as 5 miles per hr were necessary at some locations, and a limit of 50 miles per hr was in force over most of the line. Since grouting, speeds of 60 to 70 miles per hr have been allowed regardless of weather, except on a few occasions when precautionary slow orders were issued because of exceptionally heavy rainfall. Such slow orders were in force less than 24 hours, and in each case a check-up showed that the track was unaffected by the unusual rainfall.

GROUTING UNDER SUB-BALLAST TUNNEL FLOOR ON CHESAPEAKE AND OHIO RAILROAD
Grout Is Poured In by Hand and Air Pressure Is Then Applied to Inject It

The Baltimore and Ohio grouted some 3,500 ft of track in the summer of 1943. This work was in two sections, the interlocking plant at Niles Junction, Ohio, and another section at Grasselli Siding. Three roads—the Baltimore and Ohio, the Pennsylvania, and the New York Central—operate through the interlocking plant, and about 120 high-speed passenger and freight trains are handled daily. After 18 months of service, through two winters, it has been reported that the roadbed at both locations has stood up very well and apparently has been stabilized. This is indicated by a reduction in maintenance cost of about 75%. The program for 1945 provides for two grouting gangs in each of the three regions of the road, or a total of six gangs.

For a test installation, the Burlington chose a 700-ft section of single track at Salem, Nebr., the work being completed in December 1943. Here the natural ground is a black gumbo. In wet weather it becomes very sticky, liquefies when manipulated, and readily admits water to the interior of fills through the wide cracks that appear in dry weather. Test holes indicated that layers of chats, cinders, and burnt clay had been used for ballast. This had been pushed to considerable depths under the track to form troughs and basins having water-impervious bottoms and sides of black gumbo. This stretch of track had presented a really tough problem in maintenance and had required attention on the average of four days a week. In the year following grouting, this track required resurfacing only once, despite the fact that an increased volume of traffic was handled. This represents a tremendous saving, and the cost of the grouting was only $0.93 per track foot.

The Chesapeake and Ohio has solved some annoying maintenance problems in its tunnels by pressure grouting under the sub-ballast concrete floor slabs. Underflow drainage was not properly provided in some of these tunnels. The irregular contour of the excavation and porous backfill permitted water to accumulate in pockets under the floor and in some cases water flowed under the slab, carrying fine and soluble material with it. The pressure grouting was done through nipples placed in holes drilled through the floor slab along its center line, and also through the concrete gutters where this was found necessary. Maintenance has been appreciably reduced as a result of this work.

One of the most recent projects in the East is that on the New York and Long Branch Railroad at Matawan, N.J. This railroad is owned jointly by the Pennsylvania and the Central Railroad of New Jersey. The work was done by the latter, following the general practice which proved successful on an earlier experimental job done on the main line of the Central Railroad of New Jersey near Manville, N.J. Before work was started at Matawan, several test pits were dug to determine the profile of the water pockets. This proved very helpful in determining the required length of injection pipes and the depth to which they should be driven.

Pressure grouting has proved highly advantageous on special track work near terminals, where service is usually severe and maintenance is high on account of the many switches and crossings involved. Outstanding is the example of the Terminal Railroad Association of St. Louis. The grouting has been done at many points in the terminal area, and a wide range in types of fill and ballast has been encountered. All the work has been done without interruption to traffic. Included is Grand Crossing, where three tracks cross three other tracks at grade, and where an average of 750 train movements are handled every 24 hours. Since the grouting was completed, the behavior of this crossing and of the other special track work has been most satisfactory.

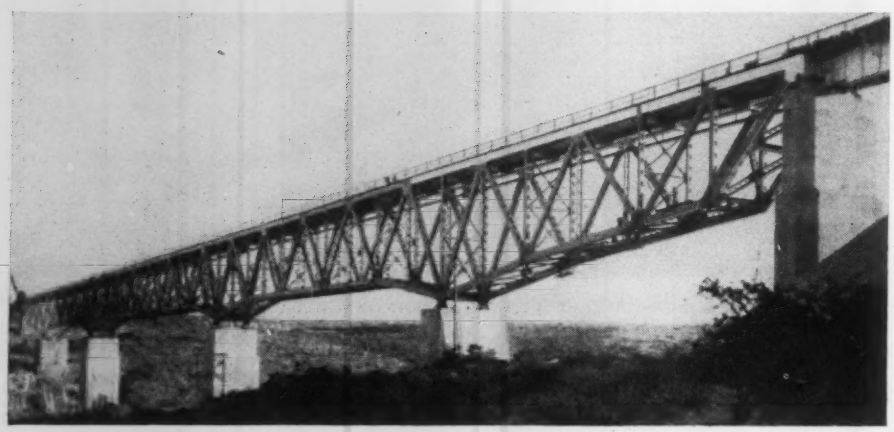
While most of the stabilization work by pressure grouting has been done by railroad forces, several railroads, including the Southern Pacific, have done it under contract. Since it is impossible to estimate in advance the amounts of material, labor, or time that will be required, there is a question as to the suitable contract basis for this type of work. On these jobs the expenses for moving the equipment and men to a job or between locations are on a cost-plus basis. When the unit has been set up, it goes on an hourly basis, which includes all equipment, repair parts, fuel, and the necessary key men, also supervision and the expenses of the superintendent for trips to the jobs. The railroad company furnishes sand and cement and the necessary laborers, who work under the direction of the foreman. Apparently this type of contract has worked out satisfactorily.

Up to the present time, some 28 major American railroads have done track grouting, and at this writing at least 7 others are getting ready to do so by preparing equipment. The roads that have done this work are continuing it and in addition have planned larger programs for 1945.

New Cantilever Carries Southern Pacific Over Pecos River

Tall, Hollow Piers Mark Design of Texas Span

By HARRY J. ENGEL, M. AM. SOC. C.E.
STAFF MEMBER, MODJESKI AND MASTERS, HARRISBURG, PA.



NEW PECOS RIVER SPAN OF SOUTHERN PACIFIC LINES
Trusses Were Erected as Cantilevers, But Joints Were Riveted to Make Spans Continuous Under Live Load and Partial Dead Load

JUST north of the Mexican border, the Southern Pacific Railroad spans the deep gorge of the Pecos River. When considering the character of the new bridge, engineers were faced with difficult erection conditions and the need for economy in the amount of steel used. Thus steel arch or simply supported spans were ruled out, and a continuous cantilever was designed and erected. Design conditions, as Mr. Engel points out, included earthquake forces and dynamic wind loadings.

WARTIME replacement of the old Pecos River Viaduct, a vital link in the Southern Pacific Railroad lines carrying transcontinental traffic across Louisiana and Texas, was considered necessary to preclude possible delays to the substantially increased war traffic.

The original Pecos River Viaduct was built in 1891. Supported on high, four-leg wrought-iron towers, it was recognized for many years as the world's highest railway viaduct. It was strengthened in 1909 by reinforcing the existing columns, and by adding a central line of columns and deck girders and trusses to the old structure. Further alterations and repairs were made in 1929 as the result of a detailed physical inspection and stress analysis, after which the structure was rated good for Cooper's E-60 loading at a train speed not in excess of 12 miles an hr. Fatigue cracks were discovered and repaired in some of the bracing details of the high towers, and the diagonal bracing rods of the towers were frequently adjusted to keep the towers plumb. Because of this situation, annual inspections were made, and the condition of the structure was each time reported upon in detail.

Under prewar traffic on this line, the speed and loading restrictions did not cause any measurable adverse effect on day-to-day operations. There was no reason to anticipate any sudden accumulation of fatigue cracks or other weaknesses requiring repairs or replacements of a character that could not be handled between trains. For these reasons the need for replacing the old structure was not considered of pressing importance before the war.

However, with the very substantial increase in the tonnage of important military traffic passing over the structure due to the war, there was some reason to anticipate an accelerated development of fatigue cracks or other weaknesses. Also, with the much more frequent train movements required, there was the danger that it might not be possible to perform repairs and replacements without delaying vital traffic movements. It was therefore concluded, during the first year of the war, that security and assurance against the possibility of serious traffic delays made it mandatory to replace this viaduct.

An application for materials was made through the Office of Defense Transportation on February 17, 1943, and an AA-3 priority was granted by the War Production Board on June 19, 1943. Construction of the new single-track bridge, which is notable for its high, hollow concrete piers and continuous-cantilever type of steel superstructure, was begun in August 1943, and completed in December 1944.

Thus the Southern Pacific Railroad now crosses the deep Pecos River gorge near Langtry, just north of the Mexican border, on an alinement about 440 ft south of the old viaduct. The site has been subject to mild earthquake shocks and is centrally located among the origins of several severe earthquakes, so that such forces had of necessity to be considered in the design. The alinement

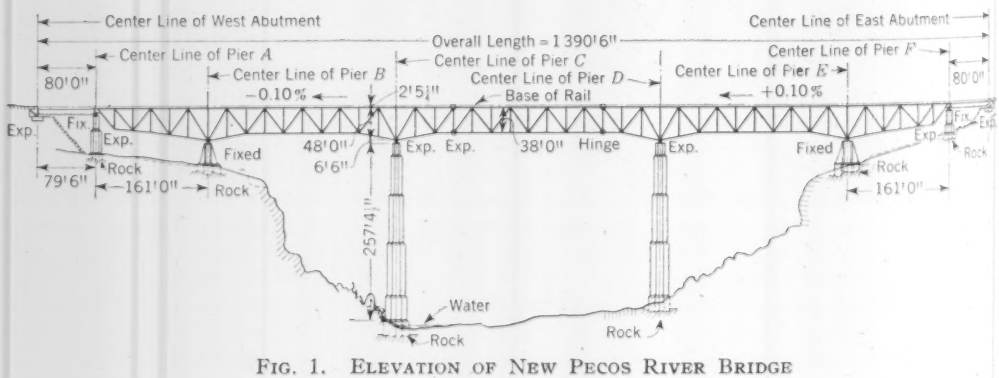
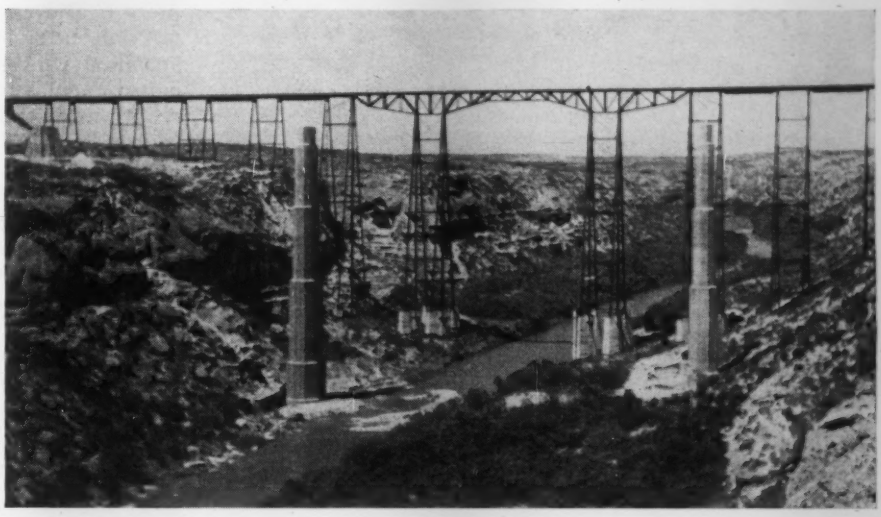


FIG. 1. ELEVATION OF NEW PECOS RIVER BRIDGE

chosen for the new crossing is the only one for a considerable distance upstream or down which is suitable for a bridge of this type. The gorge is of limestone, the soundness of which was verified by rock core drillings at each pier site. Solid rock cliffs about 770 ft apart provide excellent support for the short piers which take the longitudinal train forces. The height from normal stream level to the base of rail on the bridge is 322 ft. Long, high rock fills serve as approaches on each side of the gorge.

The design used was selected from among twelve preliminary studies as being the most economical and requiring the least amount of critical structural steel. Whereas a steel arch would have had perhaps the greatest engineering appeal, it was proved to require the most steelwork of any design studied. As for simple spans on masonry piers, viaducts on steel or reinforced-concrete towers, or an ordinary cantilever flanked by simple truss spans, these proved not to be readily adaptable to erection from the high cliffs, and they utilized materials inefficiently. The continuous cantilever deck-truss structure chosen was, on the other hand, particularly adaptable to cantilever erection from its two ends.



PIERS FOR NEW BRIDGE, WITH ORIGINAL VIADUCT IN BACKGROUND, STILL CARRYING TRAFFIC

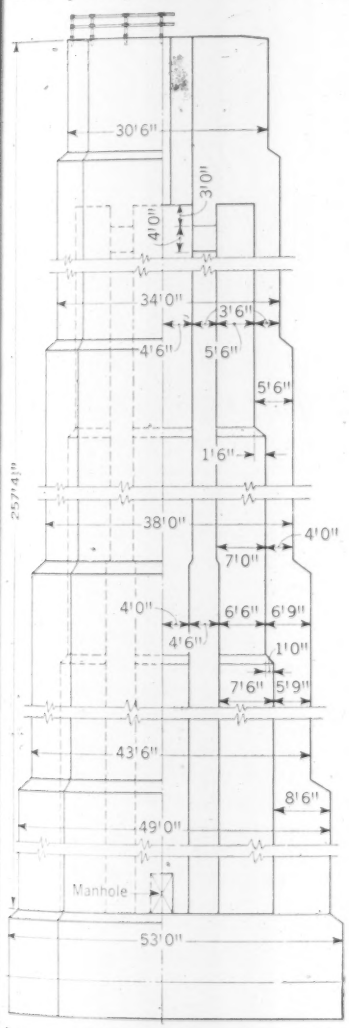


FIG. 2. HALF SECTION OF A TALL PIER—CELLULAR DESIGN USED

A 77-ft 6-in. steel girder span on each end completes the structure to the rock fills; it can be readily jacked at the abutments to counteract any settlements in the high fills.

The superstructure of the new bridge, designed for E-72 loading, is entirely of medium carbon steel with a design unit stress of 18,000 lb per sq in. in tension and bending. Horizontal earthquake forces of 0.1 *g* were used in the design. The main cantilever span is 374 ft 6 in. long, and the central suspended span is 214 ft. It is supported on roller shoes on the high center piers (*C* and *D*, Fig. 1) so as to relieve these piers of longitudinal earthquake forces from the superstructure, and tractive and braking forces from the live load. Fixed shoes on the short piers, *B* and *E*, deliver all these longitudinal forces to the cliff tops. The tendency to elastic uplift at piers *B* and *E* under live load on the cantilever span, combined with wind, is resisted by twelve anchor bolts $1^3/_4$ in. in diameter, embedded 10 ft in the concrete at each fixed shoe. These short piers are solid concrete masses of sufficient weight to prevent tension from occurring on their bases under any combination of the design loadings. The anchor piers, *A* and *F*, contain sufficient mass to resist safely the truss uplift which occurred at these points during cantilever erection of spans *BC* and *DE*, and are engaged by four upset rods of $2^1/_4$-in. diameter embedded 32 ft in the concrete at each of the end link shoes.

The two high piers (Fig. 2), were made hollow for better curing of the concrete, and also to reduce horizontal earthquake forces acting on their masses, so that the resultant reactions would always lie safely within the pier section and create reasonable unit stresses. The ordinary base pressure of dead load plus live load under the highest pier (*C*) is 9.5 tons per sq ft. The edge pressure on the pier concrete at the rock foundations under the extreme combination of dead load plus longitudinal 0.1 *g* earthquake force, plus a 15-lb per sq ft wind blowing across the gorge, is less than 450 lb per sq in. The soundness of the rock in each foundation was verified during construction by at least four drillings in each pier base, extending 20 to 25 ft below foundation level. The foundation rock is at least as strong as the concrete. For the combination of dead load plus a wind of 50 lb per sq ft blowing along the gorge on the projected area of both trusses, no tension will exist on the pier bases.

The high piers, as well as the end piers *A* and *F*, are built with vertical sides, narrowed in steps, designed to conserve form lumber, an intention which was fully realized by the slip-form method of construction employed. The ends of these piers are of octagonal shape, a motif carried also into the end features of the trapezoidal traction piers *B* and *E*. The flat pier faces and the angular junctions which result, harmonize architecturally with the rugged rock cliffs and huge boulders of the gorge.

A short, solid block of concrete caps each high pier, beneath which are the cellular sections, increasing in wall thickness from 3 ft 6 in. at the top to 8 ft 6 in. just above the base of pier *C*. There are two diaphragm walls inside each pier. Beside the normal temperature reinforcing, special bands of heavier reinforcing were placed in the outer walls at the base of each transition in pier width to resist the outward components accompanying the vertical pier forces at these sections, and there are

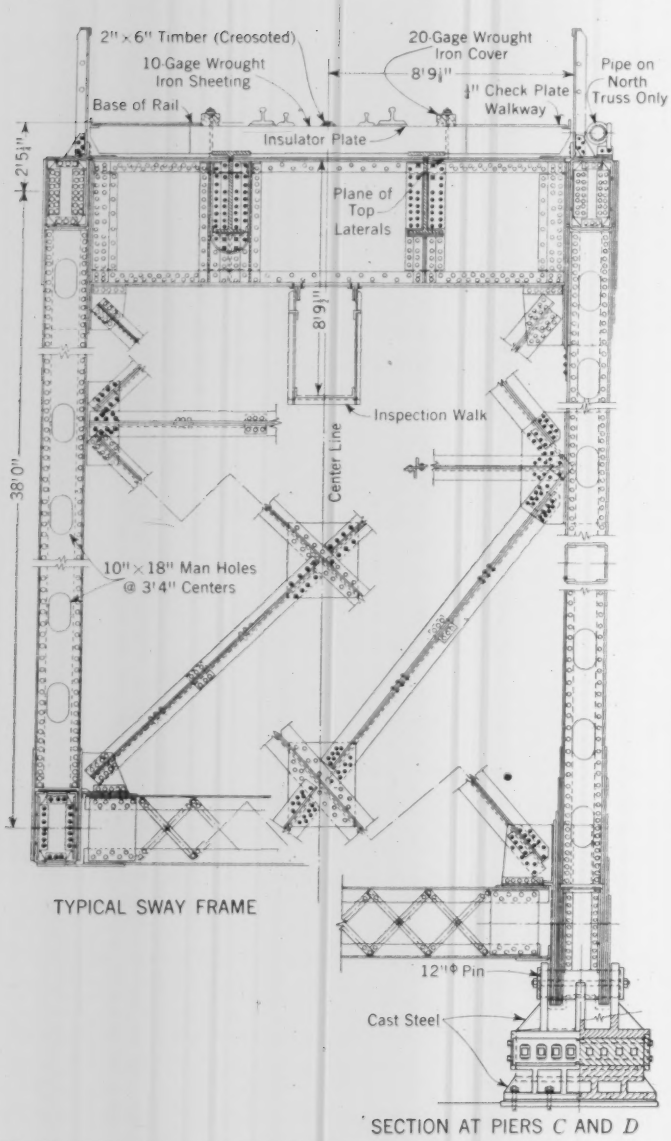


FIG. 3. COMPOSITE CROSS SECTION OF BRIDGE SHOWING SWAY FRAME AND BEARING SHOE

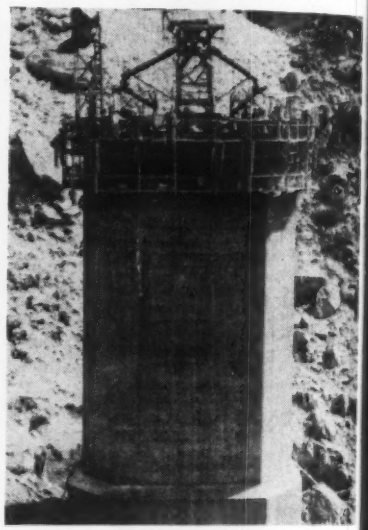
similar bands of steel in the diaphragm walls. Each pier has a solid concrete block for a base. Access to the interior of each of the two hollow piers is provided by a door at the bottom and a vertical entrance down through the pier cap, while the diaphragm walls also have access openings at top and bottom.

In view of the great height of piers *C* and *D*, and the fact that their tops must always offer reasonably exact shoe centers for the steel superstructure, a study was made of the bending effect of differential temperature on opposite pier faces with the sun shining on one side only. Assuming that a temperature differential of 50 F could exist, varying uniformly throughout the pier mass, a pier-top deflection, longitudinal to the bridge, of $5^3/_4$ in. would be obtained. But this cannot occur because in the short time that the sun shines on one side only, the temperature gradient in such a pier mass was found by analysis to penetrate hardly more than several inches beyond the surface of the concrete.

In cross section (Fig. 3), the superstructure of the bridge has trusses spaced 19 ft from center to center, established after careful studies of the overturning forces acting on the steelwork, and of the stiffness of this continuous-cantilever design. The spans are positively anchored at piers *A*, *B*, *E*, and *F*. At the roller shoes on piers *C* and *D* the structure is safe against overturning not only because the resultant of the vertical forces under specified wind loads falls inside the shoes, but also because each continuous group is supported on two pier reactions at the same level, and a third pier reaction at a higher level. The result is a structure inherently stable against overturning.

The 26-ft 9-in. panel lengths of the truss spans permit the use of rolled beam stringers and a simple system of top bracing framed into the top flanges of the stringers. Sway frames occur at the vertical columns, while the lateral loads are delivered to bottom chord bracing at the ends of the suspended span and by special vertical frames over each pier. Wind tongues in the bottom lateral system deliver the lateral loads from the ends of the suspended span to the cantilever arms.

A wrought-iron metal deck over the ties and guard rails, and checkered plate walkways on the deck, shield the steel structure from the corrosive action of brine drip and protect the timber ties against fire. An electrical stop is provided in this deck protection by a longitudinal wood strip on the bridge center line, and the tie plates are on insulation pads. The deck has steel railings, through which access may be had to ladders leading down to the pier tops and to an inspection walk which runs the length of the structure. All pier tops are protected by railings.



SLIP FORMS WERE USED ON THE TALL CONCRETE PIERS

A specific erection scheme was contemplated in the design of the superstructure, and the trusses were analyzed for it. After spans *AB* and *EF* were erected on falsework, spans *BC* and *DE* were assumed to be erected by the cantilever method to land on the high piers *C* and *D*. Erection was then assumed to be continued by the cantilever method, with the top chord joint over piers *B* and *E* only pinned and bolted, until junction would occur at the center line between piers *C* and *D*. With this junction effected, it was assumed that the suspended span would be swung. Then, with only the weight of essential steelwork and erection track on the structure, the top chord joints over piers *B* and *E* were to be freed so that no continuity would exist under this condition, and these joints were then to be reamed under no stress and riveted to make spans *ABC* and *DEF* continuous under all live load and all additional dead load. One step was varied to suit erection conditions, but design stress assumptions were precisely realized.

The Pecos River Bridge is owned by the Texas and New Orleans Railroad Company of the Southern Pacific Lines, for which R. W. Barnes, now vice-president, was chief engineer. Modjeski and Masters, engineers for the design and supervision of construction, were represented in the field by O. F. Sorgenfrei as resident engineer. The substructure, containing 15,455 cu yd of concrete, was built by Brown and Root, contractors of Houston, Tex. The superstructure, containing 2,650 tons of structural metal, was built by the Bethlehem Steel Company.

Airfields in the Pacific

Coral Used Extensively for Roads and Runways

By WILLIAM F. LUCE

LIEUTENANT COMMANDER, CIVIL ENGINEER CORPS, U.S. NAVY

THE usual description of a tropical island, with dense, malaria-infested jungles and incessant rain, makes it sound most unsuited for the waging of war, but the abundance of coral on most of the Pacific islands has in reality made them rather well suited for the rapid development of continuously usable bases. All-weather runways, parking areas, and roads, built within ten to thirty days on island after island by the Navy's Seabees, often under very adverse conditions and with limited equipment, can truly be called miracles of construction, but they were possible only with coral. In fact, coral might well be called the world's best natural material for runway construction.

Suitable coral for construction varies widely, from pockets that look and feel just like flour, to hard underwater reefs that require heavy charges of dynamite before they can be handled with a backhoe or dragline. The important criteria for good coral are purity (as little as 5 to 10% of clay or humus will often make it unstable when wet), and the presence of cementing material. The latter will leave a fine white deposit on the hands when a wet specimen is handled. Coral from which this material has been removed, such as coral sand produced by wave action, or coral that has been badly weathered, will not set up. A layer of earth and vegetation, frequently only a few inches thick, protects the coral on most islands from weathering. Coral blasted from a reef often appears to have insufficient cementing material because of the washing incidental to handling, but crushing with a sheepsfoot roller will usually produce sufficient to set it up.

RAIN DID NOT STOP WORK

The ideal sites for runways were on islands where only a few inches of earth overlay the coral. In such locations runway construction involved only removal of trees, stripping, grading, and rolling to shape. The most rapid construction was obtained by stripping deep to remove virtually all the top soil. Clean coral can be worked when quite wet; because of this, construction proceeded around the clock at one island during eight days of almost incessant rain.

The writer saw three runways where large areas had to be rebuilt because excessive foreign matter was present in the coral. The tracking in of mud by vehicles presented one of the toughest problems, especially on road construction; this was greatly aggravated by the fact that a coral island which had excellent natural drainage became a sea of mud under traffic. The admixture, however, of clean sand, particularly coral sand, does not appear to be injurious to coral.

On other islands, a subgrade was formed of sand, badly weathered coral, or clay, and covered with 8 to 10 in. of coral. Sand, of course, was the desired base, particularly in view of the heavy rains. The 46th Naval Construction Battalion completed a coral runway on a sand base requiring considerable grading in 11 days, despite rain every day. On this project coral was hauled by dump trucks from a nearby pit, spread by bulldozers and graders, given a few passes with a sheepsfoot roller,

TAXIWAYS FROM DISPERSED HARDSTANDINGS WERE ALSO CORAL PAVED

fine graded, and rolled with a 5-ton tandem roller. In addition to the rainfall, salt water was applied during rolling by improvised sprinkler trucks to produce a hard crust much like concrete.

Coral has numerous other desirable qualities. It makes a fast runway well liked by pilots. Usually it is not dusty, but if it is, a quickly laid sprinkler system along the runway can control this. Asphalt surfaces were applied in some cases. Coral drains well, and "bird baths" can be quickly eliminated by reblading or adding new material; the bond between layers is automatic, without scarifying. Bomb craters or scars from belly landings can be quickly repaired in the same way. Runways taken from the Japanese, dotted with craters 10 to 20 ft wide, have been made usable in a day or two.

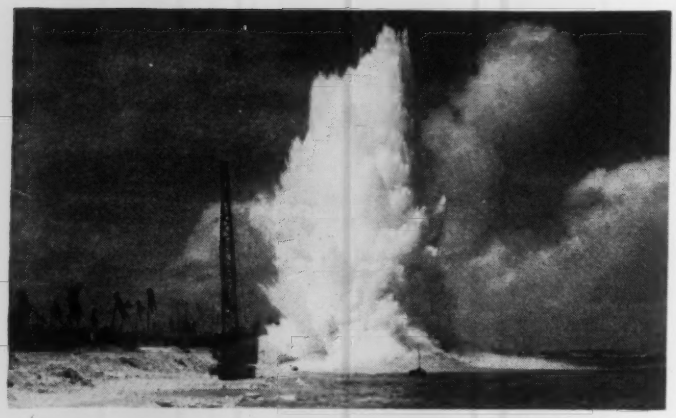
Coral seems made to order for maintaining runways with Marston-mat steel surfacing. Areas showing signs of failure are repaired without affecting flight operations by brushing screened coral through the openings in the mat. Henderson Field on Guadalcanal was made to hold up under its heavy traffic in this manner, and another important South Pacific bomber base was saved in the same way.

Coral and portland cement produce a fairly good concrete. Coral sand makes excellent fill. A marsh that could not be crossed on foot was filled by bulldozing in a fill to 2 ft above the permanent water table, and the coral runway placed on this fill showed no signs of distress.

Airfields constructed in the combat zone naturally vary markedly from those built in the United States. Certain design features, however, which proved satisfactory at advance bases, appear practicable for use elsewhere. Local conditions often require or permit major changes from normal designs.

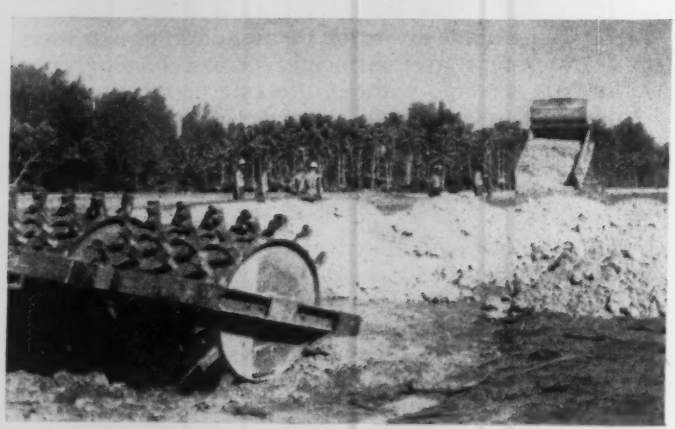
Two such conditions found in much of the South Pacific are heavy rainfall and relatively moderate winds. The former, usually in excess of 100 or even 150 in. a year, made quick surface drainage a prime factor. Runway crowns of 1.7% were found best, and a 2.0% crown on one fighter runway was never objected to by the operating personnel. Shoulder grades ranged from 1.7 to 5%, and led into flat-bottomed ditches that were drained readily, usually through deep open ditches. The relatively moderate winds, which often prevailed in one direction or two reciprocal directions, usually made it possible to build only one runway, or two or more parallel runways for large bases.

A 150-ft surface was used most of the time, and 75-ft smooth shoulders to form a 300-ft runway proved most

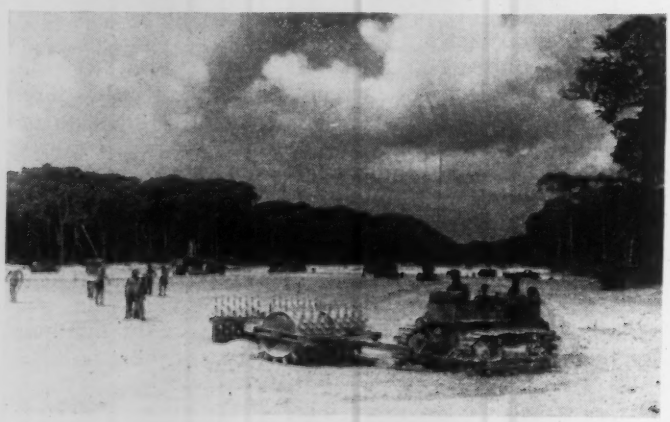


BLASTING REEF FOR RUNWAYS IN ADMIRALTY ISLANDS

EXCAVATION OF BLASTED REEF CORAL AT PONAM



CORAL TRUCKED TO AIR STRIP WAS SPREAD AND CRUSHED



PROPER EQUIPMENT SPEEDED COMPLETION OF ADVANCE BASES

satisfactory. Narrower runways seemed to result in higher operational losses, particularly at one bomber base. A cleared width of 300 ft appears sufficient for fighter planes, though 500 ft is very desirable for heavy bombers. Longitudinal runway grades as high as 1.75% have been used for fighter planes and light bombers, though 1% appears to be the maximum for medium and heavy bombers. Changes in grade should not exceed 0.3% per 100 ft, and a 0.2% maximum produces a better runway. Glide angles of 1:40 for fighters and 1:50 for bombers were provided, even when heavy clearing was involved. The provision of runover areas at each end of the runway, if only 200 to 300 ft long, will result in the saving of several planes during the life of a base.

For fighters, 40-ft taxiways, with a 100-ft width clear of all obstructions, including parked planes, should be provided. For heavy bombers, 60-ft taxiways with 175-ft clearance are needed. Taxiways should have a 2.5% crown in areas of heavy rainfall and should have 10-ft shoulders. Longitudinal grades should be limited to 2.5%, and changes in grade to 1% per 100 ft. Wide connections with runways and large areas adjacent to them for warming up and for ready planes are most desirable. A taxiway joining the two ends of the runway is essential for efficient operations, of course, and a connecting link joining the runway at a point approximately two-thirds of its length from the down-wind end will usually eliminate much taxiing after landings. A connecting taxiway between air bases located within a mile or two of each other has proved very useful at some bases, besides providing a safety factor.

SOME UNIQUE CONSTRUCTION FEATURES

Certain other features of advance military air-base construction have no counterpart elsewhere, such as clearing and heavy grading by moonlight to keep from being shelled and bombed. Land mines, grenades, duds, and such, are encountered in areas which apparently have become quite safe. Conducting the construction so as to permit emergency landings at almost any stage of the work has resulted in the saving of several planes and crews. The clearing of obstructions on the approaches should be one of the early items of construction. Since many Pacific air bases were built in groves of coconut palms, the expeditious clearing of these trees materially affected progress. As they were always planted in rows, it was possible on coral islands that had little top soil to literally mow them down by pulling a long, heavy cable behind two large tractors, taking down a row of trees at a time—usually 70 to 80 ft high. Even large hardwood trees could usually be pushed over by a bulldozer on this type of island.

The use of materials at hand was, of course, imperative. Practically all culverts under roads, taxiways, or runways, were made by welding heavy fuel drums together; they were quite satisfactory. A rather ingenious arrangement at one base, to permit bombers with exceptionally heavy loads to take off, was to extend the runway to a 75-ft bluff dropping off into the ocean. Similarity to a carrier's deck was obtained by the 87th Construction Battalion by finishing the surface as close to the bluff as possible and then blasting off the remainder.

In few activities did the Japanese seem to be as badly outclassed in the war in the South Pacific as in construction. They seldom employed heavy construction equipment, and would probably have trouble identifying some of their old bases even a few months after they were captured. One Japanese runway was partially of corduroy construction, built with palm logs.

Control of Floods at Pittsburgh Planned

Operation of 14 Reservoirs Scheduled by Corps of Engineers

By EMIL P. SCHULEEN, ASSOC. M. AM. SOC. C.E.
SENIOR ENGINEER, U.S. ENGINEER OFFICE, PITTSBURGH, PA.

DURING the years of record, the City of Pittsburgh has had an average of a little over one flood each year. Some of these have been disastrous. Studies begun by the Corps of Engineers in 1924 have been revised and now form a plan for 14 control reservoirs to be operated jointly. Eight reservoirs have been completed; the others are planned for construction when conditions permit. In this paper, which Mr. Schuleen presented at the Society's Annual Meeting, before a joint session of the Hydraulics and Waterways Divisions, the operating schedules for the reservoirs in the upper Ohio River basin are discussed.

RESERVOIR construction in the upper Ohio River basin has progressed to the point where the general effectiveness of the flood control can be determined. At the close of the calendar year 1944, there were substantially completed and in operation 4 reservoirs in the Allegheny basin, 2 in the Monongahela basin, and 2 in the Mahoning River basin (a subdivision of the Beaver basin). The last two mentioned are not effective on discharges at Pittsburgh, but are part of the reservoir system for protection of the upper Ohio Valley generally. The reservoirs in the Allegheny basin are for the purpose of flood control alone, but those in the Monongahela and Beaver basins are for both flood control and low-water regulation. These reservoirs, together with the allocation of storage in each, are described in Table I, and their general locations are shown in Fig. 1.

Certain temporary modifications have been made in the plans for operation of the Youghiogheny and Berlin reservoirs in the interest of furthering the war effort. Work on relocation of the "National Pike" (U.S. Highway Route 40) across the Youghiogheny Reservoir was suspended early in the war to conserve critical materials. This reservoir is therefore limited to operation for flood control alone, until such time as relocation of the highway can be completed. The urgent need for low-water regulation of the Mahoning River, beyond the extent originally planned, in order to provide adequate discharge at reasonably low temperatures for the benefit of busy industrial plants using the river water for cooling and condensing purposes, has made necessary a modification in the plan of operation for the Berlin Reservoir. The modified plan provides low-water regulation primarily, with seasonal variation of the flood-control storage capacity at the minimum considered necessary to furnish reasonably adequate flood protection for the Youngstown, Ohio, area. The need for considerable improvement in the low-water flow of the Mahoning River to facilitate production of war goods by plants along that stream, was demonstrated during the early years of the war, when repeated reuse of water frequently caused river temperatures to rise as high as 120 F or more. A decision regarding the ultimate plan of operation for the Berlin Reservoir has been deferred until hostilities cease.

With these exceptions, the completed reservoirs are being operated as originally intended, for the purposes indicated in Table I. It may be noted that operation of the Youghiogheny Reservoir for power generation is to be contingent upon the construction of additional headwater storage and power developments. If and when these additional projects are constructed, the allocation of storage in the Youghiogheny Reservoir, other than for flood control, will be modified to conform to hydroelectric power requirements. In the meantime, that is, after the war is over and until such time as the development of hydroelectric power may be undertaken, the storage indicated in Table I as allocated to low-water regulation will be used to improve the low-water flow of the Youghiogheny and lower Monongahela rivers in the interest of general water supply and the abatement of pollution. Low-water regulation by Mosquito Creek Reservoir is for the same general purpose as that by Berlin Reservoir.

These 8 reservoirs and the Milton Reservoir (immediately downstream from the Berlin Reservoir) provide for control of 15.8% of the drainage area above Pittsburgh and 13.7% of the area drained by the Ohio River above Wheeling (Lock 12), W.Va. These percentages do not include 158 sq miles of area tributary to Pymatuning Reservoir, owned and operated by the Commonwealth of Pennsylvania for flood control and low-water regulation, nor do they include areas tributary to reservoirs owned and operated by other public or private agencies for water supply or generation of hydroelectric power. With the exception of Pymatuning Reservoir, however,

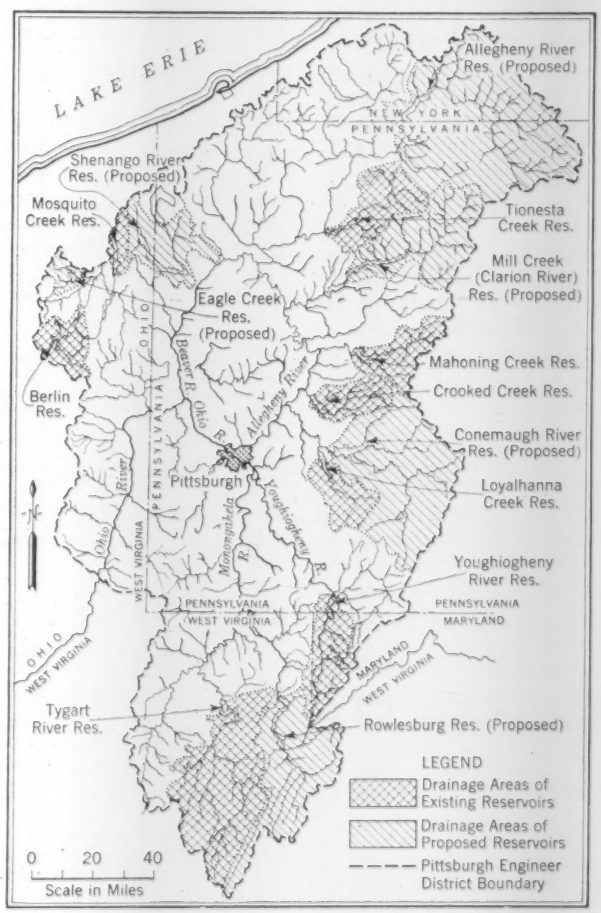


FIG. 1. EXISTING AND PROPOSED RESERVOIRS IN THE UPPER OHIO RIVER BASIN

these non-federal developments are not operated to provide dependable flood control, although in some instances they do have a regulating effect on stream flow.

In addition to the projects already completed, studies have been made of six other reservoir possibilities to serve as a basis for expanding the system. These reservoirs, together with the proposed allocation of storage for various purposes in each, are also described in Table I, and their general locations are shown in Fig. 1. With the exception of that on the Conemaugh River, all the additional reservoirs would be for the dual purpose of flood control and low-water regulation, or of flood control and hydroelectric power development.

Low-water regulation by the Allegheny River Reservoir would result in abatement of pollution in that stream and improvement in the quality of the river water for domestic and industrial use. The Mill Creek Reservoir on the Clarion River and the Rowlesburg Reservoir on the Cheat River would be for the purpose of hydroelectric-power development in addition to flood control. Both these reservoirs are parts of comprehensive hydroelectric-power plans involving construction of additional storage and head developments in the tributary basins.

The Shenango and Eagle Creek reservoirs, in addition to flood control, would provide low-water regulation for the Shenango and Mahoning rivers, respectively. The former project in this respect would supplement low-water regulation by Pymatuning Reservoir, and the Eagle Creek project would supplement regulation by the Berlin, Milton, and Mosquito Creek reservoirs. The effectiveness of all the reservoirs in the Mahoning and Shenango basins would extend throughout the length of the Beaver River, which is formed by the junction of those two streams.

The reservoirs on the Conemaugh River, Allegheny River, Shenango River, and Eagle Creek are included in the general comprehensive plan for flood control and other purposes in the Ohio River Basin, as approved by Congress. Construction of the Rowlesburg Reservoir has been recommended by the Chief of Engineers, U.S. Army, but the Mill Creek Reservoir is still under consideration. If all these reservoirs are constructed, the percentage of drainage area controlled will be extended to 43.6% of the area above Pittsburgh and 37.4% of the area above Lock No. 12 on the Ohio River. It should be emphasized, however, that funds to initiate actual construction work have not been allotted for any of these proposed additional developments. The extent to which they may be included in the ultimate reservoir system is therefore open to question at this time.

Detailed, coordinated operation schedules have been prepared for each of the completed reservoirs, and are now being used in their operation to serve Pittsburgh and the upper Ohio Valley generally. Similar detailed schedules have not been prepared for any of the six possible additional developments. Operation plans for these have been limited to such approximations as were found necessary to permit a fair estimate of the economic value of the various projects.

The detailed, coordinated plans for operating the reservoirs now in use are composed essentially of five separate schedules, designated as follows:

Schedule A—Routine Operation
Schedule B—Minor Rises
Schedule C—Flood Control
Schedule D—Water Supply Storage
Schedule E—Release of Water Supply Storage

Schedule A provides for the routine passing of flow through the reservoir when storage or release of water is not required for any specific purpose. This schedule is applicable primarily to those reservoirs serving to provide only flood control, or to those periods of the year when that function alone is intended. When low-water regulation also is one of the reservoir purposes, routine operation is essentially absorbed in Schedules D and E. Schedules B and C are designed to provide a basis for the control of excess stream flow, while Schedules D and E establish the procedure for storage and release of water for low-flow regulation, when the latter functions are a part of the reservoir purposes.

The same general pattern has been used in the development of schedules for all the completed reservoirs. However, the controlling conditions and the details of operating instructions have been designed to fit the particular requirements of each individual reservoir. In each case, the governing conditions are stated as specific values of time, stage, or elevation, and the corresponding operating instructions are stated as specific degrees of opening for the needle valves or gates in the outlet control works. The number of the particular valve or gate to be used is also given, to provide as far as possible for symmetry of flow in the stilling basin when more than one outlet is in use.

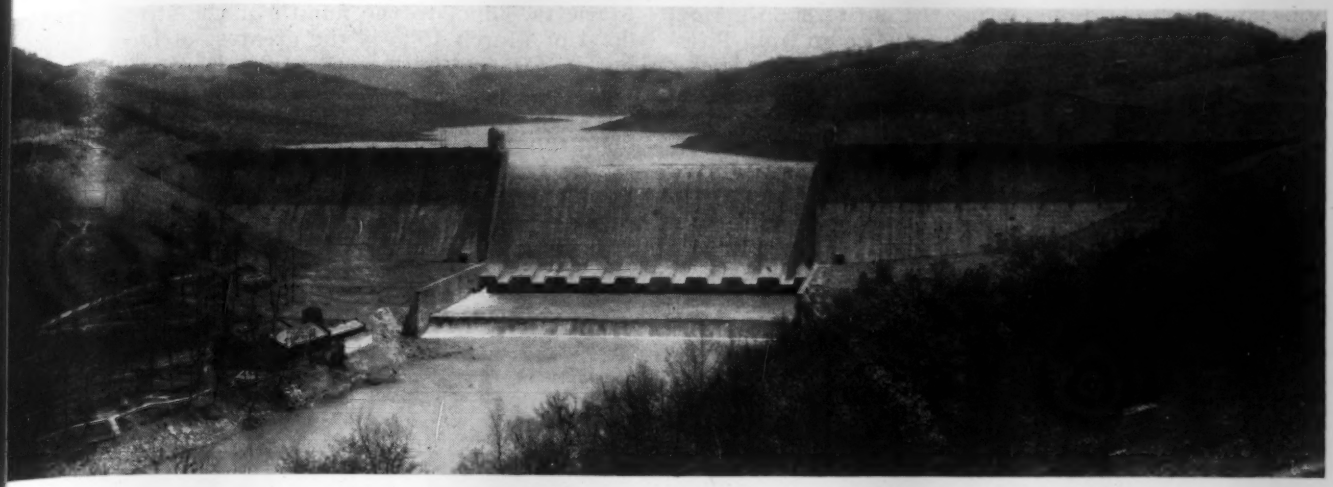
SCHEDULES ARE FLEXIBLE

Although the various schedules are written in a firm and commanding style, they are intended primarily as guides in the exercise of judgment. This is true especially of Schedules A, B, and C. Obviously, it would be impossible to incorporate into a concise schedule all the many factors that must be considered in providing effective reservoir operation. Deviations from the strict terms of the schedules therefore are permitted when it is evident that better results will be obtained thereby.

Schedule C is designed to provide a basis for operating during general floods of all magnitudes up to and including what might be termed the "reasonable maximum." Should a flood of greater proportions occur, particularly in the basin local to a specific reservoir, operation according to the general flood-control schedule might result in the premature filling of this reservoir. A special schedule has therefore been prepared for use under such circumstances. However, since this schedule also serves as a basis for operation by the dam tenders during an emergency, it has been included in "Emergency In-

RESERVOIR	STREAM	BASIN	PURPOSE
Existing:			
Tionesta Cr.	Tionesta Cr.	Allegheny	FC
Mahoning Cr.	Mahoning Cr.	Allegheny	FC
Crooked Cr.	Crooked Cr.	Allegheny	FC
Loyalhanna Cr.	Loyalhanna Cr.	Allegheny	FC
Tygart R.	Tygart R.	Monongahela	FC, LW
Youghiogheny R.	Youghiogheny R.	Monongahela	FC, LW, WP
Berlin	Mahoning R.	Beaver	FC, LW
Mosquito Cr.	Mosquito Cr.	Beaver	FC, LW
Proposed:			
Conemaugh R.	Conemaugh R.	Allegheny	FC
Allegheny R.	Allegheny R.	Allegheny	FC, LW
Mill Cr.	Clarion R.	Allegheny	FC, WP
Rowlesburg	Cheat R.	Monongahela	FC, WP
Shenango R.	Shenango R.	Beaver	FC, LW
Eagle Cr.	Eagle Cr.	Beaver	FC, LW

Legend: FC = flood control; LW = low-water regulation; WP = hydroelectric power; E = earth dam of selected rolled fill; C = concrete gravity dam; U = uncontrolled; G = gate controlled



(1) TYGART RIVER DAM DISCHARGING FROM NEEDLE VALVE NO. 2, AT LOWER LEFT OF SPILLWAY SECTION
Similar Needle Valve Is Located at Extreme Right, but the Eight Outlets Between Are Slide-Gate Controlled

structions to Dam Tenders" rather than among the general schedules.

The "Emergency Instructions to Dam Tenders" are considered necessary in the event that all forms of communication between the District Office and a dam are disrupted, although this possibility is extremely remote. In addition to regular telephone service, each dam is provided with radio transmitting and receiving equipment, and with a generator set driven by a gasoline engine to furnish emergency power. Nevertheless, it has been considered advisable to take the precaution of providing dam tenders with instructions for use should such an unforeseen emergency arise.

A general outline of the personnel organization for operation and maintenance of the various reservoirs will facilitate an understanding of the emergency instructions furnished to the dam tenders. Each dam has three attendants, two of whom are provided with residences on the premises. These three employees, in addition to performing all necessary operating functions, do all the ordinary maintenance work in connection with the dam and its appurtenances. Normally all three dam tenders work on the daytime shift, but their number permits the assignment of one man to each of three shifts during periods of flood or other emergency.

All operations having to do with storage or release of water are made only on specific orders from the headquarters of the Flood Control Section in the District Office, except as provided for in the emergency instructions. This headquarters group in the District Office is composed of a small number of engineers. In addition to certain other functions, it is charged with the duties of planning, coordinating, and directing the operation and maintenance of all reservoirs and other flood-control works.

Considering the detailed nature of the emergency instructions, the question might well be asked, "Is it necessary for dam tenders to be provided with instruc-

TABLE I. EXISTING AND PROPOSED RESERVOIRS IN THE UPPER OHIO RIVER BASIN

		Drainage Area			Allocation of Reservoir Storage						
							FC		LW or WP		
Type of Dam	Spillway Type	Sq. Mi. at Dam	% Above Pittsburgh	% Above Wheeling (Lock 12)	Season	Dead Acre-Ft	Acre-Ft	In.	Acre-Ft	In.	Total Acre-Ft
E	U	483	2.5	2.0	All	7,800	125,600	4.9	...	..	133,400
C	G	341	1.8	1.4	All	4,500	69,700	3.8	...	..	74,200
E	U	278	1.5	1.1	All	4,500	89,500	6.0	...	..	94,000
C	G	291	1.5	1.2	All	2,000	93,300	6.0	...	..	95,300
C	U	1,183	6.2	4.8	Winter	11,200	278,400	4.4	...	..	289,600
					Summer	11,200	178,400	2.8	100,000	1.6	289,600
E	U	435	2.3	1.7	Winter	5,000	150,000	6.4	99,000	4.3	254,000
					Summer	5,000	100,000	4.3	149,000	6.4	254,000
E & C	G	275	..	1.1	Varies	1,800	11,500 to 116,500	0.8 to 7.9	105,000	7.1	118,300
E	U	97	..	0.4	Winter	5,000	33,000	6.4	69,000	13.3	107,000
					Summer	5,000	22,000	4.3	80,000	15.4	107,000
		Subtotal	15.8	13.7							
C	G	1,351	7.1	5.5	All	4,000	270,000	3.8	...	..	274,000
C	G	2,190	11.4	8.9	Winter	20,000	910,000	7.8	195,000	1.7	1,125,000
					Summer	20,000	585,000	5.0	520,000	4.5	1,125,000
E	U	833	4.4	3.4	Winter	90,000	222,000	5.0	544,000	12.2	856,000
					Summer	90,000	148,000	3.3	618,000	13.9	856,000
C	G	941	4.9	3.8	Winter	147,000	251,000	5.0	492,000	9.8	890,000
					Summer	147,000	167,000	3.3	576,000	11.5	890,000
C	U	434	..	1.7	Winter	14,000	113,000	4.9	...	..	127,000
					Summer	14,000	88,000	3.8	25,000	1.1	127,000
E	U	95	..	0.4	Winter	7,000	33,000	6.5	59,000	11.6	99,000
					Summer	7,000	22,000	4.3	70,000	13.8	99,000
		Subtotals	27.8	23.7							
		Totals	43.6	37.4							

tions covering operations during the entire flood period, when it would be possible for someone to go from Pittsburgh to the dam in a few hours to take charge of operations?" The latter is exactly what would be done under the circumstances, but it should be remembered that these instructions would serve also as a guide to the District Office personnel in operating the reservoir during a flood of extreme proportions.

In Table II are outlined the effects that each of the existing or porposed reservoirs had, or would have had, on the floods of December 1942, January 1937, and March 1936 at Pittsburgh and Wheeling. The overall effects of the existing and proposed reservoirs on these floods at the same localities are also shown graphically, in Fig. 2.

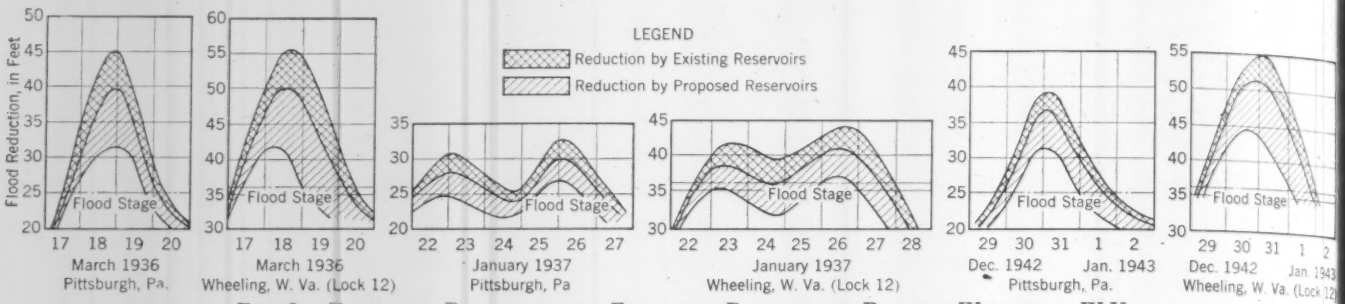


FIG. 2. EFFECT OF RESERVOIRS ON FLOODS AT PITTSBURGH, PA., AND WHEELING, W.VA.

The flood of December 1942 is the greatest that has occurred at Pittsburgh since the construction of the reservoir system was sufficiently advanced to provide appreciable benefits. At the time of this flood the Tionesta, Mahoning, Crooked Creek, Loyalhanna, and Tygart reservoirs were substantially completed and in full operation. Construction of the Youghiogheny and Berlin reservoirs was advanced to the extent that they were able to function as detention basins with all available outlet facilities fully open. Mosquito Creek Reservoir had not yet been begun and therefore had no effect on the flood runoff.

The January 1937 flood is the greatest that has occurred during modern times on the Ohio River from about Marietta, Ohio, to the mouth of the stream; and the flood of March 1936 is the greatest of record at Pittsburgh. During both of these floods, the Tygart Reservoir was in process of construction but was not sufficiently far along to have more than a deterring effect on the flood runoff.

An examination of Table II will disclose that reductions by the various reservoirs may not be in the same relative proportion for all floods. This results from variations in the distribution of the effective rainfall and snow melt. For example, during the March 1936 flood, a considerable portion of the runoff affecting peak stages at Pittsburgh and Wheeling came from the Kiskiminitas River. The reservoirs on Loyalhanna Creek and the Conemaugh River, both tributaries of the Kiskiminitas, therefore would have provided more than their normal share of the total reduction during that flood.

TABLE II. FLOOD REDUCTIONS, IN FEET, AT PITTSBURGH AND WHEELING DUE TO RESERVOIRS

	Pittsburgh, Pa.			Wheeling (Lock 12), W.Va.		
Reservoir	Dec. 31, 1942	Jan. 26, 1937	Mar. 18, 1936	Dec. 31, 1942	Jan. 26, 1937	Mar. 19, 1936
Existing:						
Tionesta Cr.	0.7	0.7	0.6	0.8	0.7	0.6
Mahoning Cr.	0.5	0.3	0.9	0.6	0.3	0.9
Crooked Cr.	0.5	0.2	0.7	0.6	0.3	0.7
Loyalhanna Cr.	0.4	0.3	1.3	0.5	0.3	1.3
Tygart R.	0.5	0.8	1.2	0.6	0.8	1.3
Youghiogheny R.	0.2*	0.3	0.8	0.2*	0.4	0.8
Berlin	..	..	..	0.2†	0.3†	0.0
Mosquito Cr.	..	..	..	0.1	0.1	0.0
Sub-total	2.8	2.6	5.5	3.6	3.2	5.6
Proposed:						
Conemaugh R.	2.2	0.6	4.6	2.6	0.6	4.6
Allegheny R.	1.2	1.0	0.6	1.5	1.2	0.6
Mill Cr. (Clarion R.)	0.9	0.8	1.9	1.1	0.8	1.9
Rowlesburg (Cheat R.)	1.1	0.7	1.2	1.3	0.8	1.2
Shenango R.	..	..	..	0.5	0.6	0.0
Eagle Cr.	..	..	..	0.1	0.1	0.0
Sub-total	5.4	3.1	8.3	7.1	4.1	8.3
Total	8.2	5.7	13.8	10.7	7.3	13.9

* Would have been doubled had reservoir been completed and in full operation.

† Joint effectiveness of Berlin and Milton reservoirs, less effectiveness of Milton Reservoir operated alone.

GENERAL EFFECTIVENESS OF RESERVOIR CONTROL

The hydrographs of Fig. 2 indicate that even with the 14 existing and proposed reservoirs in operation, floods would not be completely eliminated. These reservoirs, however, would fully control all floods except those of great magnitude and would so reduce the latter as to render them of far less consequence. Should other flood-control reservoirs be added, or should the proposed storage and head developments for power be constructed in the Clarion, Cheat, and Youghiogheny basins, floods of great magnitude would be even further reduced in intensity. In a similar manner, should the Lake Erie–Ohio River Canal be constructed, with the proposed large summit reservoir on the Grand River in Ohio, flood waters from the Mahoning River above Youngstown would be diverted into that reservoir, and the result would be complete elimination of floods in the Mahoning River Valley and considerable further reduction in flood crests at Wheeling.

Unfortunately, definite action regarding reservoir construction in the upper Ohio basin was taken so late in the general development of the area, that most of the otherwise good reservoir sites had already been occupied by railroads, highways, oil and gas pipe-lines, electric transmission lines, and general industrial and community developments. These advances have so restricted reservoir location that in many cases construction costs are prohibitive, or development of the site to provide full control of the flood runoff is not economically feasible.

It is probable, therefore, that the ultimate reservoir plan, even if expanded to include projects for flood control and other purposes in addition to the 14 existing and proposed reservoirs described in Table I, will not completely eliminate all floods. In fact this probably cannot be done economically by storage alone. Residual flood damages, however, would be relatively small, and in most cases could, if desired, be eliminated by the construction of local flood-protection works.

Open Caissons for Ohio River Dam Successfully Sealed

By H. P. Winn, Assoc. M. Am. Soc. C.E.
Major, Corps of Engineers, Army Service Forces, Pittsburgh Engineer District

CONSTRUCTION of Montgomery Dam, on the Ohio River 32 miles below Pittsburgh, involved the use of open caissons as foundation shells. This method was chosen by the contractors, Booth and Flinn Company, as being less expensive than alternative

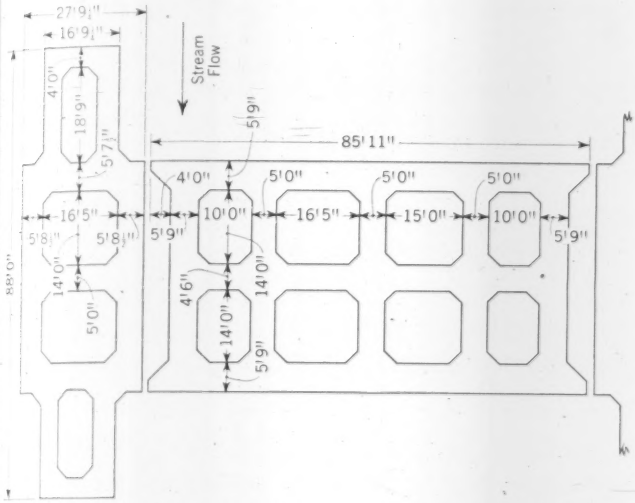


Fig. 1. Plan of Typical Pier and Sill Caissons

procedures. The prime problem was to seal the shells after sinking to bedrock, thus assuring a dry foundation.

The entire structure of the dam, consisting of eleven piers, was built upon the caissons. Separate caissons were sunk for piers and gate sills, as shown in Fig. 1. A space of about a foot between caissons at the narrowest point provided working room. At all other points at least 5 ft of space was left to provide room for dredging.

During construction of the foundation caissons, two cofferdams enclosed the working area. The first closed the abutment on the north end of the dam, together with six piers, and included the gate sills. When progress permitted, the remaining section of the dam was enclosed and the river diverted through the completed part.

The upstream and outstream arms of the first cofferdam were composed of a double row of steel sheet piling, while the inside face of the lower arm was of 4 by 12-in. timbers placed vertically on the river bed. With the exception of the upper and lower outstream corners, none of the piling was driven to rock. The second cofferdam differed from the first in that the inside row of sheeting on the upper arm was driven to rock and the outstream arm was also on rock. The downstream arm consisted of a single row of steel sheet piling heavily bermed on both sides. In both cofferdams, the inner wall of the upstream arm was not more than 7 or 8 ft from the upstream face of the piers.

After the area within the cofferdam was dewatered, it was leveled off at an elevation about 25 ft above rock. Four-foot lengths of 4 by 12-in. timber were then placed side by side at right angles to the line of the cutting edge of the caisson, and on these the half I-beam cutting edge was laid with the flange vertical (Fig. 2). Next, the forms for the first lift were placed and heavily blocked. Collapsible well forms used for the openings in the caissons eliminated much trouble in setting and removing.

Ten days after the first lift was poured, the second was placed, and five days later the caisson was sunk to a depth not exceeding 12 ft. The third lift was then placed. When the forms were removed (in three to five days, according to weather conditions), the caisson was sunk the remainder of the distance to rock. When the two adjacent caissons had been landed on rock, interlocking steel sheet piles were driven across the space between them on the upstream and downstream sides.

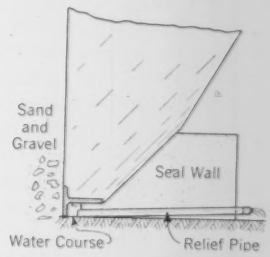
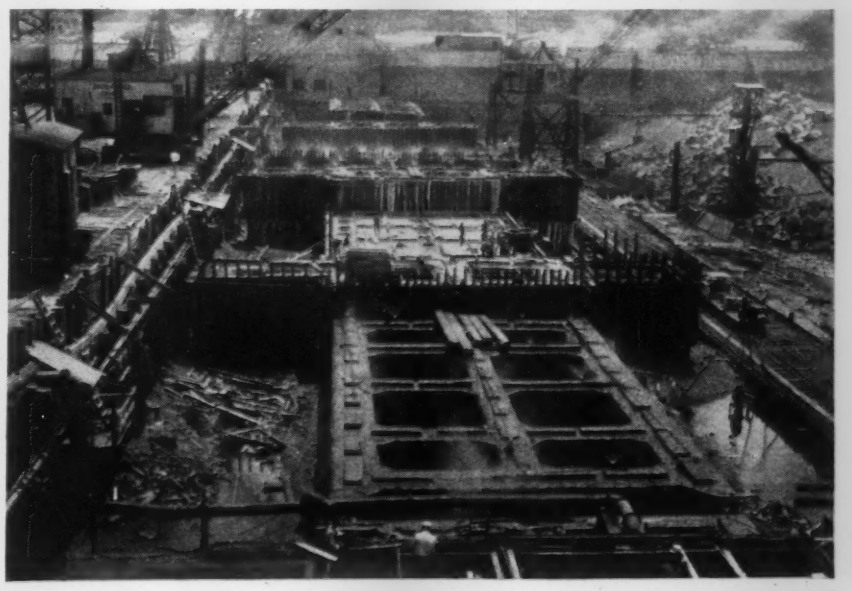


Fig. 2. Section Through Cutting Edge of a Caisson

Seating of the cutting edge was accomplished by chipping out the rock inside with a pneumatic jackhammer, or in certain instances by light blasting. When the caisson was finally seated, it was embedded about 3 in. in rock and made to bear fairly evenly at all points.

After the overburden had been excavated and the rock surface given a preliminary cleaning, all faulty material was removed. A keyway 5 ft deep and 8 ft wide was excavated in rock throughout the entire length of the dam, joining one that was left under the river wall of the locks at one end of the dam, and the abutment



Concrete Caissons Were Sunk to Bedrock While Three Lifts Were Poured

wall at the other end. At appropriate points, pump sumps 10 or 12 ft in diameter and 7 or 8 ft deep were blasted in the rock.

Leakage in the cofferdam and its proximity to the structure made the use of pumps necessary at all times. As soon as a caisson was sunk to rock, the water naturally followed down the side through the loosened sand and gravel and appeared under the cutting edge, generally on the upstream side. Even after final seating, enough was present in the caisson to present a problem. Where the rock was badly laminated, a certain amount of water worked in through the seams.

To prevent leakage under the cutting edge, a small concrete seal wall, 2 ft by 2 ft, was poured along each cutting edge where necessary. The water thus confined was relieved through pipes and led out of the caisson by a pipe or a covered trench. Any other seepage not confined by the seal wall was conducted in the trenches along the bottom of the keyway to a point just inside the limits of the pour, where a neat cement dam held an outlet pipe.

The upstream and downstream well of each of the pier caissons was poured by the tremie method. After the caisson had been pumped out and the rock foundation of these two wells approved, the entire caisson was allowed to refill with water. Since the piers were the first to be sunk to rock, this filling did not present a difficult problem, as the caisson was entirely surrounded by sand and gravel and the water in no way interfered with the lowering of the sill caissons on each side. Tremieing was done through a 10-in. pipe attached to a hopper suspended from a derrick, so as to be adjustable in height. Ten feet were poured in this way. This method proved very successful. After a reasonable curing period, the water was pumped out and the two remaining wells were poured in the dry in the usual way.

GROUTING OF CAISSON

Grouting of the trenches and relief pipe was a separate feature. A system of about seven pipes from an entire pier or sill caisson was led into an adjacent open well. Mounted on the caisson over this well was a pneumatic concrete blower of 1-cu yd capacity.

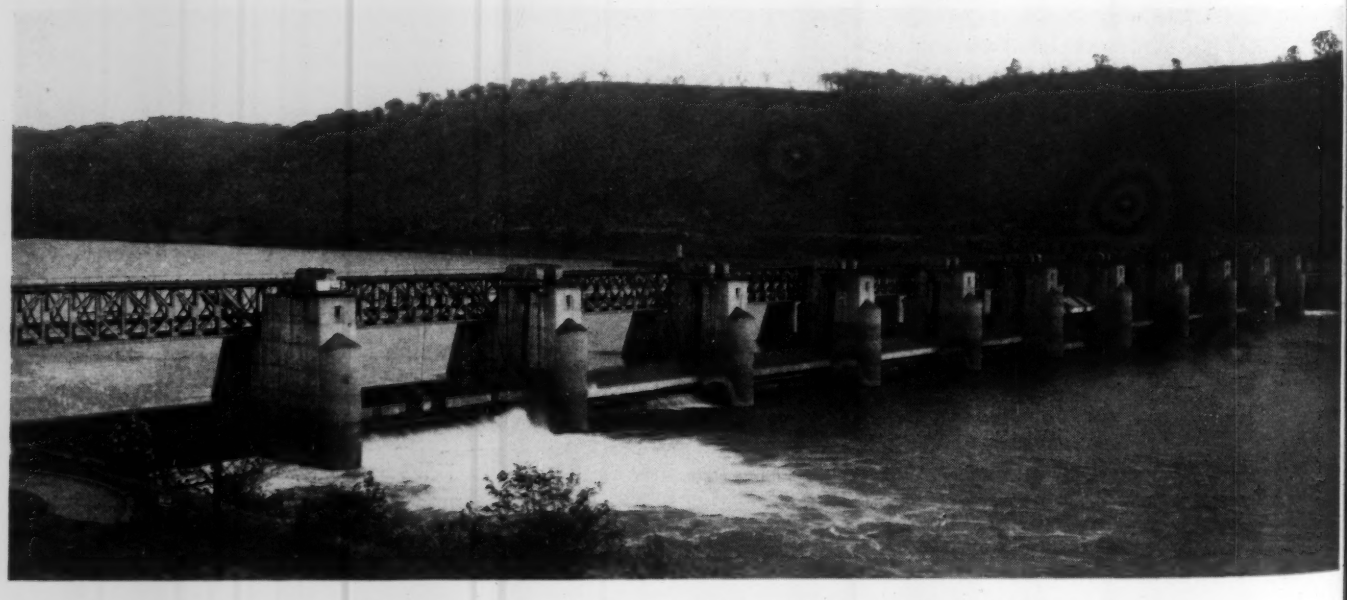
In combating a water head, it is necessary that grout be forced into the system rapidly and in as large a volume as possible. A "blower" was better adapted to this than an ordinary grout machine. At all locations but the last four, grout for the blower was mixed at the central plant and conveyed in regular concrete buckets. The buckets were scaled, and a 4-in. valve was inserted at the bottom through which grout could be drained. Although a sand-cement grout was used at first, neat cement was finally adopted as being more satisfactory. To avoid lumpiness, all grout was strained through a screen of $\frac{1}{4}$-in. mesh. Often all wells but one in a sill caisson were grouted on a single system, and the last well, in which the pump sump had been located, was thereby made quite dry. Any water remaining was confined in a hole covered by sheet iron and removed during concreting by a pump outside the working area. The hole and embedded pipe were then grouted.

Once grouting of a pipe was begun, it was never halted until completion. After each line, with its vent pipes, was blown out with air, a comparatively thick mixture of grout was forced through. As the operations proceeded, the vent-pipe valves were gradually closed. When all the valves were about 90% closed and the last air and water had escaped, the valves sealed themselves.

When no more grout could be forced through the machine, the hose was disconnected and blown out to be sure that the stoppage was not due to clogging. Then a small pneumatic grout machine of the paddle-mixer type was connected to each vent pipe and an attempt was made to force more grout into the system. When this had been done, the valves were completely closed and the grout allowed to set for an hour or so before the valves were removed for cleaning. The texture of the grout mixture depended a great deal on the conditions encountered. Investigation revealed that very little grout was "lost" outside the caisson. This was primarily due to the fact that only enough pressure was applied to overcome the hydrostatic head.

Normally, in trench grouting, a pressure of 10 to 20 lb was sufficient at the start. This was increased to 100 lb, or a maximum of three times the hydrostatic head. The nature of each particular system governed procedure.

In view of the good results obtained on this project, the same method was tried with equal success on another navigation dam subsequently constructed on the Allegheny River.



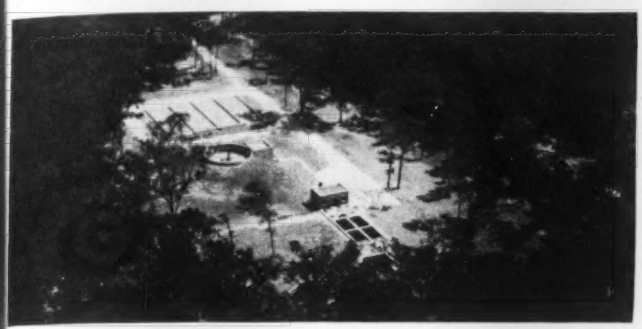
MONTGOMERY DAM ON OHIO RIVER, FOUNDED ON OPEN CAISSONS

Development Board Coordinates Georgia Public Works

By H. J. Friedman, M. Am. Soc. C.E.

Consultant, Public Works Panel, Agricultural and Industrial Development Board of Georgia, Brunswick, Ga.

AUTHORITY to plan an ordered and comprehensive development of the State of Georgia and its resources was given to the Agricultural and Industrial Development Board of Georgia upon its creation by the 1943 state legislature. It is the purpose of the Board not to duplicate the work of any existing governmental agency, but to coordinate and supplement the activities of all agencies and institutions engaged in the state's development. It was realized that the people desire something more than plans. The Board's basic function is, therefore, to stimulate action on the various programs.



Sewage Disposal Plant on St. Simon's Island, Ga., Constructed with Lanham Act Funds

Typical of Projects Scheduled by Georgia Public Works Panel

Membership of the Board is representative of all sections of the state and varied lines of endeavor. There are 21 members. Five are the heads of the State Departments of Agriculture, Conservation, Education, Health, and Public Service. A sixth is the chairman of the Board of Regents of the university system or a member designated by the chairman. The remaining 15 are appointed by the Governor from the state at large—leaders in agriculture, business, finance, industry, and labor.

The Board is divided into seven three-man panels, each responsible to the chairman for an individual phase of activity. These panels are Agriculture, Education, Government, Health, Industry, Public Works and Trade, Commerce and Business. The active administration of all operations is the responsibility of an appointed executive director. The operation of the Board is best illustrated by reference to functions of one panel—in this case, the Public Works Panel.

This panel, which began work May 1, 1944, operates through a director, with consultants and a supporting staff. A program covering the following features was adopted:

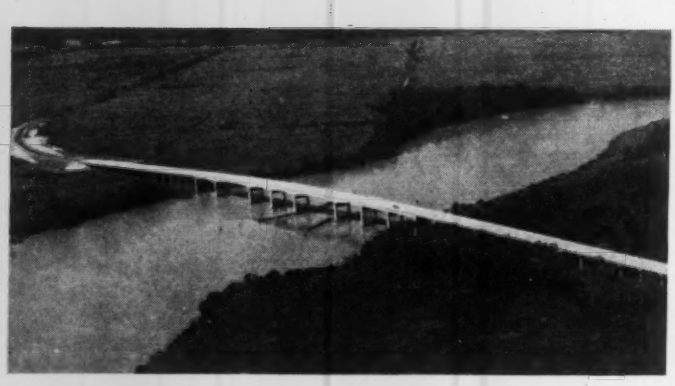
1. Build up a technical library to take advantage of all available research and surveys.
2. Divide the state, by urban and rural areas, into integral units in which public works programs may be logically developed.
3. Aid in preparing drafts of legislation covering highway rights of way, limited access, control of roadsides, planning and zoning, airport development, legalizing of reserves for public works, control of ground waters, establishment of a state port authority, and other pertinent legislation.
4. Work toward coordinating allocation and expenditure of funds that may be made available by Congress to the different federal agencies for postwar planning and construction.
5. Using the arterial highway system as a base, sponsor arrangements for a master plan and long-range program of public improvements for each of the integral urban units.
6. Prepare master plans and public improvement programs for each integral rural unit, using the Public Works Panel personnel supplemented by the services of the different state and federal agencies available.
7. Arrange, with the aid of the interested federal and local agencies, for a coordinated program of airport construction, including preparation of design standards for airports for small communities that can be built cheaply and quickly.

The panel approach to public works planning in the State of Georgia is a modification of the unit area analysis method originated by the National Resources Planning Board. The 159 counties of the state have been divided into 55 unit areas consisting of one to four counties each. There is a considerable saving in time and money through following this procedure. Of the 55 areas, 43 are classed as rural. In these areas there is no city with a population of over 15,000.

No study is undertaken unless requested by the appropriate governing body, usually the county commissioners or the city council. This is in line with the general policy of the Board that development programs in any field can succeed only with the support of the local community. The rural planning program is under way in several areas. The field work on the program for the first area requesting the service has been completed and the report published, covering a six-year program of public works. Also, 56 other counties and 60 cities have made application, and resolutions are pending in some 66 counties and 75 municipalities. Financial and school studies have been made in approximately 30 of the counties that have passed resolutions.

A definite procedure in planning a public works program in rural areas is followed. A representative of the panel is first assigned to the area to study the finances of the counties and municipalities. His prepared state-

A Health Center Completed at Columbus, Ga., by the Federal Works Agency



SOUTH ALTAMAHA RIVER BRIDGE CARRIES THE COASTAL HIGHWAY
Built with State and Federal Aid

ment provides the basis for an estimate of funds available for the public works program. State, federal, and local officials then meet with representatives of the panel to determine local needs. All recommendations and data are integrated and a program prepared for each of the counties and municipalities in the area. The resulting report is then submitted to the local governmental group. The only charge borne by the counties and municipalities is the actual cost of publishing the reports.

It is proposed to provide a continuing service in connection with these studies. It will be desirable to revise the programs yearly in the light of new developments. Current information is being furnished the cities and counties regarding state and federal funds that may be made available for public works. Guidance and aid are being given in the processing of applications and plans through the appropriate state and federal agencies.

In each of the 12 urban areas containing counties with cities of over 15,000, it is proposed to prepare a long-range program of public works. A comprehensive study of the arterial highway system is the initial step. These surveys are to be financed by the State Highway Department and the U.S. Public Roads Administration. The highway study is to be followed by a survey by a consultant group to determine a comprehensive long-range program of community facilities, including airports, water supply, sewage disposal, recreational areas, housing, and public buildings. It is expected that these studies will be financed through local and state aid, with such federal assistance as Congress may make available for postwar public works planning under Title V of the War Reconversion Act of October 3, 1944. The studies will include definite proposals for financing, including rights of way and other land acquisition.

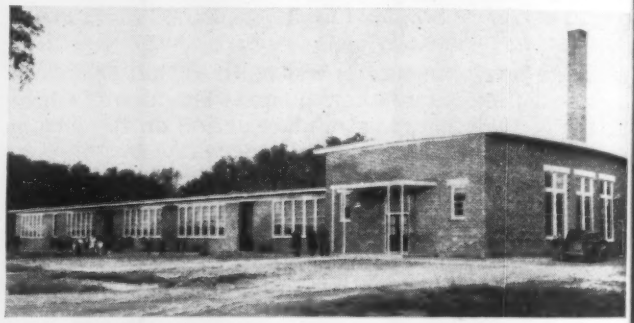
Considerable progress has been made in carrying out these plans through the cooperation and collaboration of the State Highway Department with the Public Roads Administration. Studies and recommendations covering arterial highway facilities in the Savannah and Brunswick areas have been completed by qualified consultants. A similar comprehensive study is being made in the metropolitan Atlanta area and the Waycross area.

Early in the work of the Public Works Panel it became apparent that the passage of certain acts by the state legislature was essential. In cooperation with the State Highway Department and the Public Roads Administration, bills covering different phases of right-of-way acquisition and control were prepared. During the 1945 session of the legislature five panel-sponsored bills were passed. An act eliminating limitations on the width of right of way, permitting such widths as required, was one of these. Other acts passed set up a State Port Authority, authorized the Brunswick Port Authority, legalized reserves for public works, and authorized the adoption of a plane coordinate system of land description. It is anticipated that other bills will be passed by the 1946 session of the legislature, including control of highway access and roadsides, providing for urban parking facilities, planning, and zoning.

The Public Works Panel is also cooperating with the Civil Aeronautics Administration and state authorities in establishing a program of airport construction. Legislation for establishing a State Aeronautic Department, based on the Uniform State Aviation Code, has been prepared, as well as an act for the control of land uses and clearances in areas adjoining airports, following the Model Zoning Act of the Civil Aeronautics Administration. It is expected that the passage of this legislation will also be secured at the 1946 session of the state legislature to permit taking advantage of any federal aid that may be made available for airport construction.

A policy of providing a Class 1 airport or air park, at an estimated average cost of $18,000, in each smaller county in the state, has been adopted.

Comprehensive studies of proposed state port development are nearing completion. A study of the state's water resources, under the direction of the Board's Rivers and Harbors Committee, has been completed. It includes all the major river systems of the state and provides a master plan of improvement covering water power, navigation, flood control, water supply, soil erosion, and pollution abatement. Legislation is being prepared to implement the program through a State Water Resources Authority. An action program based on the recommendations in this report has been prepared.



PUBLIC SCHOOL BUILDING BUILT AT BRUNSWICK, GA., WITH FEDERAL FUNDS

Interest in the study is indicated by a recent arrangement requested by the City of Atlanta and Fulton County for a supplementary detailed report on water supply and navigation in the Chattahoochee River in the Atlanta area. The cost of this additional report is being borne by the city and county as well as by the state.

The state port development study is also being handled by the Rivers and Harbors Committee. Georgia has two major seaports—Savannah and Brunswick. The two ports, while only 80 miles apart, requested a cooperative study which the state agreed to undertake, using funds contributed by the two communities as well as by the state. The studies are comprehensive, and include an investigation of industrial potentialities, land and water freight rates, trade possibilities with Latin America, and design of proposed state dock facilities.

The State Port Authority Act, passed by the 1945 legislature, authorizes the issuance of $15,000,000 in revenue bonds for the two ports, and provides for a three-man authority. The State Port Authority has been appointed. A report on the proposed development will be issued soon and provide a program for the Authority.

Drainage of Clay Strata by Filter Wells

A Brief Explanation of the Theory Involved and Its Application

By K. Terzaghi, M. Am. Soc. C.E.

Consulting Engineer and Lecturer, Graduate School of Engineering, Harvard University, Cambridge, Mass.

During the last decade, several dams and fills failed on account of time lag in the consolidation of underlying strata of soft clay. Owing to the low permeability of the clay, very little of the excess water was able to drain out of the clay during the construction of the fills. As a consequence, the shearing resistance of the clay remained almost as low as it was at the outset, and the fill failed by spreading as soon as the shearing stress in the clay became equal to the shearing resistance.

The increase in the shearing resistance of the clay can be accelerated by the introduction of vertical filter wells, which serve as supplementary outlets for the excess water. In order to accomplish the desired effect at a minimum expense, it is necessary to investigate the effect of the spacing of the wells on the average rate of consolidation and to choose the spacing such that about 80% of the consolidation will occur during the construction of the fill. A very much larger spacing is inadequate and a very much smaller one is uneconomical. The theory of the influence of the spacing of the wells on the rate of consolidation is highly mathematical and very involved. However, once the fundamental equations have been solved, the required information can be obtained without any computation, by the use of ready-made graphs as here described.

Determining Rate of Consolidation

In Fig. 1 (*a*) is shown a section through an embankment resting on the horizontal surface of a stratum of soft clay. In order to accelerate the consolidation of the loaded stratum, filter wells have been established which permit the escape of part of the excess water in a horizontal direction into the wells, which in turn deliver it to a filter bed located between the clay and the base of the fill. The rest of the excess water flows in an upward direction from the clay into the filter bed. The layout of the system of wells appears in Fig. 1 (*b*). The vertical sections indicated by dashed lines divide the consolidating layer of clay into prismatic blocks. Within each block drainage proceeds as if the vertical sides of the block were lined with an impermeable membrane, because the excess water in the clay located on either side of the section escapes in opposite directions. The problem of computing the rate of drainage can be simplified without appreciable error by assuming that each block is cylindrical. On this assumption the flow is symmetrical about the axis of the block (Fig. 1, *c*).

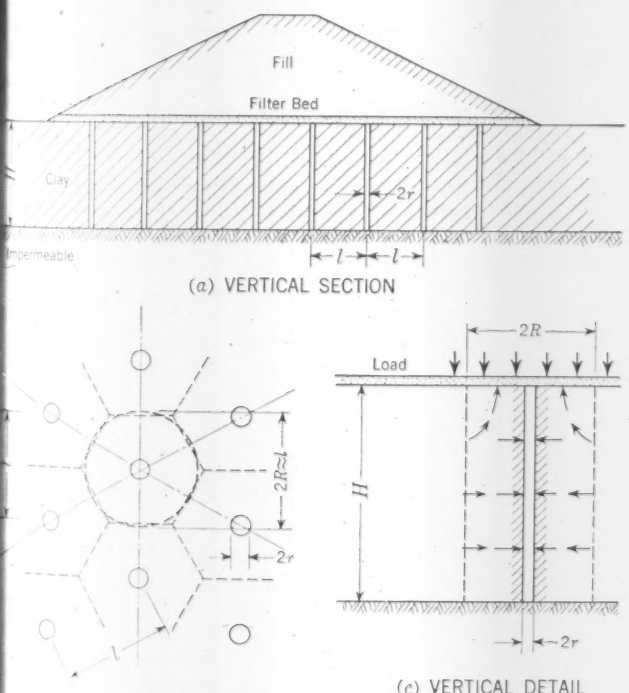


Fig. 1. Arrangement of Filter Wells Under an Embankment

Within the block the flow of water proceeds as if the cylindrical surface and the base of the block were covered with an impermeable membrane. Let

H = thickness of the layer of clay

R = radius of the cylindrical blocks, roughly equal to $\frac{1}{2}l$, the spacing of the wells

r = radius of the wells

$n = R/r$ = ratio between the radius of the blocks and the wells

k_r and k_z = coefficients of permeability of the clay stratum in radial and vertical directions

m = coefficient of volume decrease of the clay, equal to the decrease of the volume of the clay per unit of volume due to an increase of the load on the clay by unity

γ_w = unit weight of the water

t = time

$c_r = \frac{k_r}{\gamma_w m}$ and $c_z = \frac{k_z}{\gamma_w m}$ = the coefficients of consolidation for radial and vertical directions

U = average degree of consolidation of the clay stratum at time t, equal to 100 times the ratio between the quantity of water which flowed out of the clay before time t and the total quantity of water which leaves the clay under the influence of the weight of the embankment

U_r = average degree of consolidation at time t, if the water could escape only through the walls of the wells

U_z = average degree of consolidation at time t, if the water could escape only through the upper surface of the clay

The values of the soil constants k_r, k_z, and m can be determined approximately by means of soil tests on undisturbed samples in the laboratory. The mathematical relation between the degrees of consolidation U_z, U_r, and U was derived in 1940 by N. Carrillo ("Simple Two and Three Dimensional Cases in the Theory of

Consolidation of Soils," *Journal of Mathematics and Physics*, 1942). It is given by the equation,

$$100\% - U = \frac{1}{100}(100\% - U_z)(100\% - U_r) \ldots (1)$$

The differential equation for computing the value U_z was published by the writer in 1924. The solution can be represented by a function,

$$U_z = f_1(T_z) \ldots (2)$$

wherein T_z is a pure number, called the time factor, equal to $(c_z/H^2)t$. At any given time t, the degree of consolidation U_z merely depends on the time factor T_z, regardless of the thickness of the stratum and the mechanical properties of the clay. By plotting the values U_z against the values T_z, the curve C_z in Fig. 2 is obtained.

The differential equation for computing the value U_r was derived and solved in 1934 by L. Rendulic ("Der hydrodynamische Spannungsausgleich in zentral entwässerten Tonzylindern," *Wasserwirtsch. und Technik*, 1935) in connection with an experimental investigation which was carried out in the writer's soils laboratory in Vienna. The solution can be represented by a function,

$$U_r = f_2(T_r) \ldots (3)$$

wherein T_r is a pure number, called the time factor for radial drainage, equal to $(c_r/4R^2)t$. At any given time t, the degree of consolidation U_r depends solely on the value of the time factor U_r and on the value of the ratio $n = R/r$. In Fig. 2 the plain curves C_{10} and C_{100} represent the relation between U_r and T_r for $n = 10$ and 100. These curves supersede those on page 292 of the writer's *Theoretical Soil Mechanics*, (Wiley, 1943). Credit is given to the U.S. Engineer Office, Providence, R.I., for calling attention to the error in computing these curves.

Quite recently R. A. Barron solved the problem of drainage towards wells in clay for boundary conditions other than those which were considered by his predecessors ("The Influence of Drain Wells on the Consolidation of Fine-Grained Soils," report prepared by the Soils

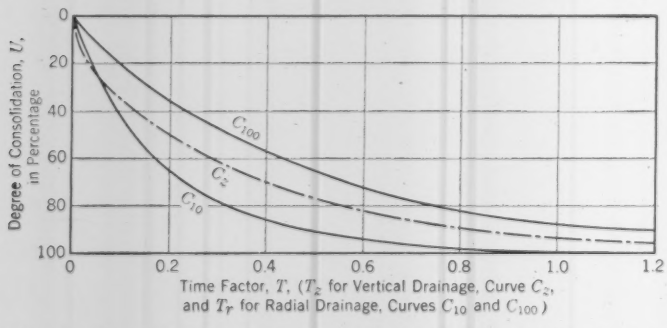


FIG. 2. RELATION OF DEGREE OF CONSOLIDATION, COEFFICIENT OF CONSOLIDATION, AND TIME FACTOR—FOR BOTH VERTICAL AND RADIAL DRAINAGE

Laboratory of the U.S. Engineer Office, Providence, R.I., July 1944). He represented his findings in a set of graphs which permit utilization of the results without any further computation.

A PRACTICAL EXAMPLE

The following numerical example illustrates the use of graphs in connection with drainage-well problems. The depth H of the clay stratum in Fig. 1 (*a*) is assumed to be 20 ft. The coefficient of permeability of the clay is the same in every direction, $k_z = k_r$; whence $c_z = c_r = c$. The wells have a diameter of 1 ft and they are spaced 10 ft in both directions. Replacing the vertical prismatic blocks which surround the wells by cylindrical blocks with equal horizontal cross-sectional area, we obtain for the diameter of these blocks approximately $2R = 10$. The problem is to determine the average degree of consolidation of the clay stratum at the time t, when the average degree without wells would be $U_z = 20\%$.

From curve C_z (Fig. 2) we obtain for $U_z = 20\%$, the value $T = T_z = 0.032$. Introducing this value into Eq. 2, we get

$$T_z = 0.032 = \frac{c}{H^2}t, \text{ whence } t = \frac{0.032H^2}{c} \ldots (4)$$

The time factor T_r for radial flow at time t (Eq. 2) is

$$T_r = \frac{c}{4R^2}t = \frac{c}{4R^2}\frac{0.032H^2}{c} = \frac{0.032 \times 20^2}{4 \times 5^2} = 0.128$$

Since $R/r = 10$, the degree of consolidation U_r at time t is equal to the ordinate of that point on curve C_{10} in Fig. 2, whose abscissa is equal to 0.128, or $U_r = 49\%$. Introducing the values $U_z = 20\%$ and $U_r = 49\%$ into Eq. 1,

$$100\% - U = \frac{1}{100} \times 80 \times 51 = 41, \text{ or } U = 59\%$$

Hence the presence of the wells increases the degree to which the clay stratum consolidates within time t from 20% to 59%.

In order to compute the time t, it would be necessary to determine by means of laboratory tests the values k and m, which appear in the equation for the coefficient of consolidation c. Since $c = k/\gamma_w m$, we obtain from Eq. 4 the following value of t,

$$t = 0.032H^2\frac{\gamma_w m}{k}$$

TEST RESULTS

In order to verify the theory which is represented by the curves C_{10} and C_{100} in Fig. 2, the following test was made in the writer's laboratory in 1934 by L. Rendulic (see previous reference). A cylindrical specimen of plastic clay with an external diameter of $2R = 5$ cm, and a height of 8 cm, was introduced into a watertight container. The cylindrical surface of the specimen was covered with a watertight membrane. The central part of the specimen was occupied by a cylindrical body made out of a sand-mica mixture whose compressibility was about equal to that of the clay. This body represented a cylindrical drainage well with a diameter of $2r = 0.8$ cm. After the specimen was installed in the chamber, the chamber was filled with liquid which was under constant pressure.

Since the two ends of the test specimen were in contact with metal disks, the consolidation of the specimen occurred only by drainage in horizontal, radial directions towards the central well. The water which came out of the well was collected and weighed and the discharge was plotted against time. This experiment was repeated at five different pressures. Thus five different time-discharge curves were obtained. Then the values of k and m for the clay were determined by independent tests and the time-discharge curves were computed. As a result of these tests, it was found that the agreement between the theoretical and the experimental curves is even more satisfactory than the agreement between theoretical and experimental curves for drainage in a vertical direction.

Performance of TVA Structures Studied

Gates, Spillways, Stilling Basins, and Sluices

By G. H. Hickox, M. Am. Soc. C.E.
Senior Hydraulic Engineer, TVA, Norris, Tenn.

RECORDS accumulated during the years of operating the TVA's hydraulic structures have shown that in general these structures are satisfactory. However a number of hydraulic phenomena have been observed for which there is at present no ready explanation. Mr. Hickox hopes that attention drawn to these phenomena may bring to light similar observations by other observers, as well as explanations of the phenomena in question. His paper was presented before the Annual Meeting of the Hydraulics Division of the Society.

CONSTRUCTION activities of the Tennessee Valley Authority have centered chiefly around large dams. In practically every case, despite attempts to duplicate design wherever possible, unusual conditions have been presented. It was therefore expected that certain irregularities in performance of the hydraulic appurtenances might result. Considerable study by TVA engineers has shown that practically all the structures have proved entirely satisfactory. The records have accumulated for all types of hydraulic elements—some revealing unusual behavior. In particular, attention is directed to the spillway gates, sluices, and stilling basins operating with both spillway and sluice discharge.

CREST GATES OF THREE TYPES

Three types of crest gates are used by the Authority on its dams. Norris Dam, designed by the Bureau of Reclamation, is equipped with drum gates on the crest. These gates are operated by floating them in a chamber filled by water admitted from the reservoir. In their lowered position they have the same shape as the crest and offer no obstruction to the flow. When raised, however, water falls from the downstream edge of the gate to the face of the spillway. For small discharges, with the gate raised several feet, the nappe vibrates. The chief effect of this vibration is the formation of waves on the face of the spillway, giving it a sort of washboard appearance. The vibration of the nappe does not appear to be dependent on the admission or exclusion of air from the space beneath the nappe, since it occurs when the nappe is fully aerated. It may be caused by vibration of the gate itself. It does not continue for high discharges.

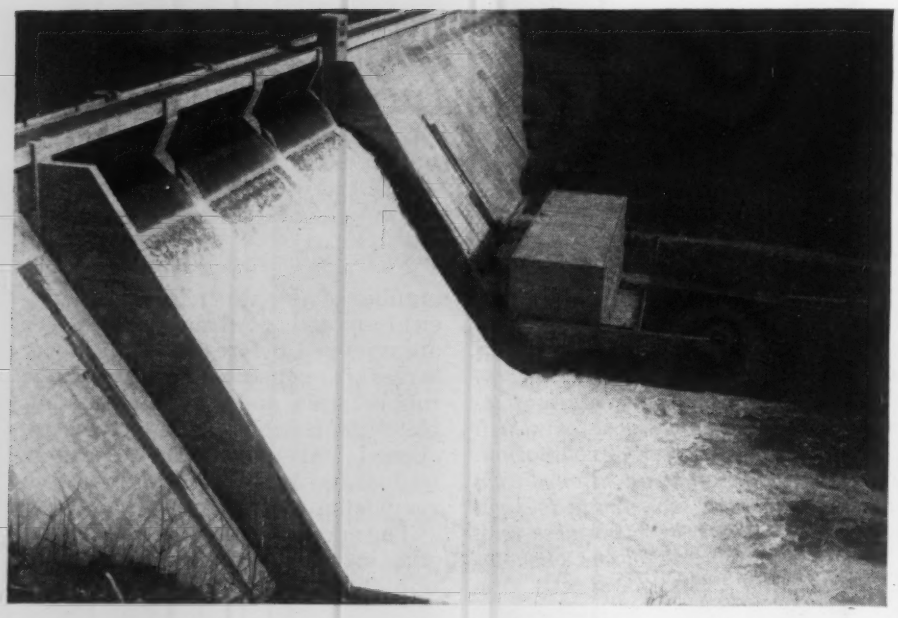
As the discharge over the raised drum gates increases, air is drawn beneath the nappe and finally escapes through the sheet of water on the face of the dam. The mixture of air and water results in a thickening of the sheet on the face of the dam, which is easily visible as a ridge or roll near the crest. The air escapes by passing upward through the sheet of water from the face of the dam to the surface. The roll on the face of Norris Dam, and the turbulence caused by the escaping air at its lower boundary, are shown in an accompanying photograph. High-speed motion pictures of this phenomenon show that the air rises to the surface in white puffs not unlike cotton bolls.

At Kentucky, Pickwick Landing, Guntersville, and Chickamauga dams the crest gates are of the vertical-lift type with two leaves operating in the same vertical plane. Moderate discharges are passed by raising a number of the upper leaves a sufficient amount. All the upper leaves are removed before any of the lower leaves are opened. These gates have rubber seals and are almost completely watertight. When they are opened and closed, the discharge begins and ends abruptly, with a very definite cutoff.

The operation of these double-leaf gates introduced the problem of aeration of the spillway nappe. When the upper gate is raised, discharge occurs over the lower leaf, removing the air from the space beneath the nappe and creating a vacuum. In order not to overload the lower leaf by excessive reduction of pressure, it was necessary to admit air to the space beneath the nappe. At Pickwick Landing Dam this was accomplished by means of two 16-in. pipes embedded in the piers and terminating in the pier faces just below the upper edge of the lower gate leaf. Measurements of the air actually required by these gates were made at the dam to verify the results of model tests. At one time during the tests the pressure beneath the nappe was reduced 9.7 ft below atmospheric. This was the limit allowed by the gate designers as the maximum safe load. With this reduction of pressure, the nappe was depressed considerably, although there was a head of almost 20 ft of water over the lower leaf. A complete description of the determination of air requirements for this type of gate and the measurements at Pickwick Landing Dam was published in an article by the writer, "Aeration of Spillways" (in Transactions, Am. Soc. C.E., 1944, Vol. 109, pp. 537–566).

The third type of gate, which has been used on Watts Barr, Fort Loudoun, Cherokee, Douglas, Hiwassee, Apalachia, and Ocoee No. 3 dams is the conventional

Temporary Sluice Outlet at Nottely Discharging 2,000 Cu Ft per Sec



AIR DRAWN BENEATH THE NAPPE AT NORRIS DAM ESCAPES THROUGH THE SHEET OF WATER ON THE SPILLWAY

radial gate. These gates are also provided with rubber seals and very little leakage has been experienced. Their operation is not unusual and requires no comment.

Stilling basins for dissipating the energy of spillway and sluice discharge at the Authority's dams were developed and thoroughly tested in the hydraulic laboratory. In general, different types of aprons were required for spillway and sluice discharge, and where both kinds of discharge occurred on the same apron, the design finally adopted was usually a compromise between conflicting requirements. Aprons for dissipating the energy of spillway discharge may be divided generally into two types—those on which a hydraulic jump can be produced and those on which it cannot.

Norris Dam is an example of a high-head dam where the hydraulic jump is utilized for dissipating the energy of spillway discharge. The Norris stilling basin is a horizontal apron a few feet below the river bed and joined to the face of the spillway by a gentle slope. A steep sill provides the transition from the paved apron to the natural river bed. A profile of Norris Dam spillway and apron is shown in Fig. 1. The apron was designed for a maximum flood discharge of 240,000 cu ft per sec. To date the maximum spillway discharge experienced has been 30,000 cu ft per sec. The performance of the apron was entirely satisfactory.

The hydraulic jump has also been used on some of the low-head dams of the Authority where the tailwater conditions were favorable. This is the case at Watts Bar Dam, illustrated in Fig. 1. The apron is short, horizontal, and at the elevation of the river bed; it has a sill at the end with a sloping upstream face that serves to reduce the necessary amount of paving. The maximum spillway discharge at this dam since its completion has been 92,000 cu ft per sec. Agreement with the results of model tests was excellent.

In general, the satisfactory operation of the hydraulic jump in dissipating energy requires that the discharge on the apron be distributed uniformly over its length. In the case of dams with many spillway gates, such distribution of discharge is not always practical. In many cases it is desirable to open a few gates fully before opening the remainder. When this happens the tailwater depth for a given discharge per gate increases as later gates are opened. Since the hydraulic jump is effective over a relatively small range of tailwater depth, other methods of dissipating energy must be devised.

Pickwick Landing is an example of a dam of this type. There are 22 spillway gates, 8 of which have been designated as control gates. The upper leaves of these gates, spaced at regular intervals across the spillway, are raised gradually until they are completely removed before the remaining gates are opened. The tailwater depth below these gates for the initial opening of any one gate varies from that for zero discharge to that for the discharge with 21 gates open, a range of approximately 200,000 cu ft per sec in discharge, and about 30 ft in tailwater depth for normal river conditions. Operation of Kentucky Dam downstream increases this range still further.

Because of the extreme range of tailwater, the hydraulic jump was not feasible except for a very limited range of operating conditions. The apron finally selected for Pickwick Landing Dam was a short horizontal one with a simple sill at the end, having a sloping upstream face. This sill, which is 5 ft high, directs the high-velocity water upward toward the surface, removing it from the bottom and protecting the bed from harmful scour. A profile of this dam (Fig. 1) shows the tailwater range expected. Discharges of 300,000 cu ft per sec have passed Pickwick Landing Dam since its construction. Careful soundings below the apron fail to disclose any evidence of erosion or undermining. The apron is completely satisfactory from the standpoint of erosion but causes surface waves that persist for some distance below the dam and inter-

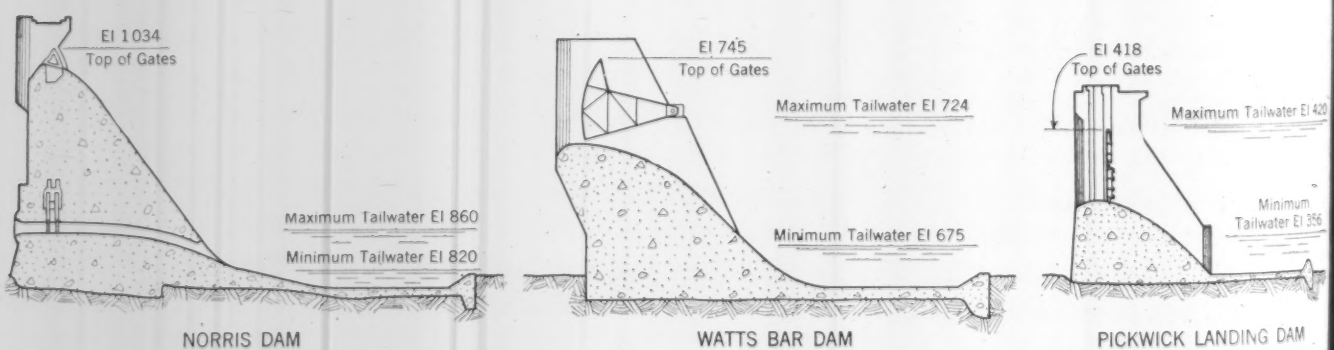
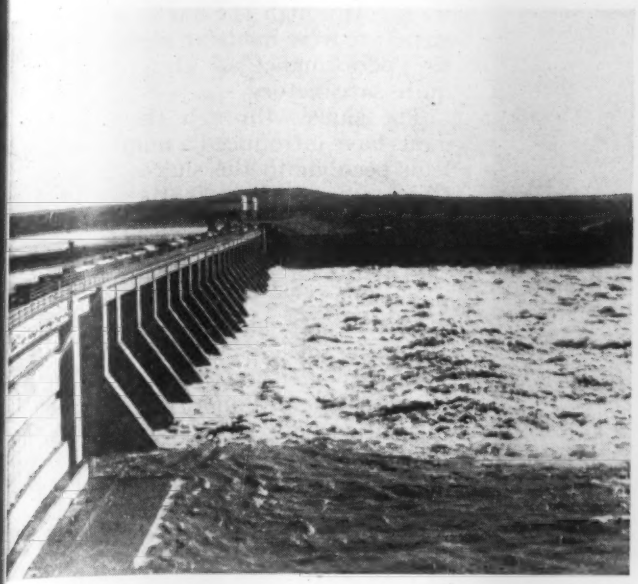
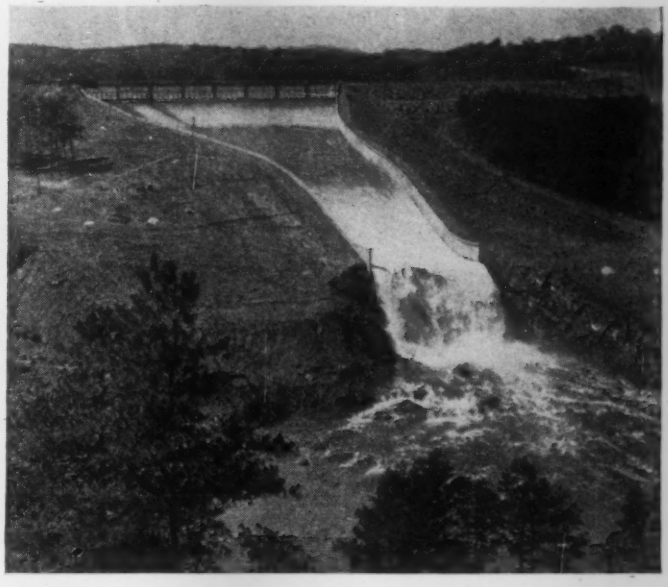


FIG. 1. PROFILES OF NORRIS, WATTS BAR, AND PICKWICK LANDING DAMS ILLUSTRATE VARIOUS TREATMENTS OF STILLING BASINS



PICKWICK LANDING SPILLWAY PASSING 250,000 CU FT PER SEC —NOTE SURFACE ROUGHNESS ON APRON



SPILLWAY AT NOTTELY PASSING 2,400 CU FT PER SEC—ITS CAPACITY IS 48,000 CU FT PER SEC

fere somewhat with navigation. For this reason, it was necessary to provide structures to reduce the wave heights in the navigation channel approaching the lock from downstream.

The spillways for Nottely and Chatuge dams are quite different from those of any of the other main-river or tributary dams. The dams are earth and rock-fill structures and have separate spillways. The spillways are placed in saddles in the reservoir rims and discharge down the hillsides into the river below the dams. They are similar in size, having crests 300 ft long, and operate under maximum heads of 11 to 13 ft. The plan of the spillway crest is an arc of a circle. Curved training walls, perpendicular to the spillway crest line, concentrate the discharge into a long, relatively narrow chute.

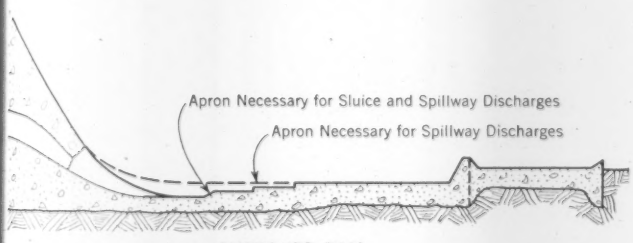


FIG. 2. APRON FOR CHEROKEE DAM SHOWING MODIFICATION FOR SLUICE DISCHARGE

At both dams this chute terminates on the hillside above the river bed. The high velocity of the flow down the chute is depended upon to carry the discharge past the toe of the bank and into the river bed. These spillways were designed for a maximum discharge of approximately 40,000 cu ft per sec. Although the maximum discharge experienced at Nottely Dam has been only 2,400 cu ft per sec, and that at Chatuge Dam 3,000 cu ft per sec, the flow in the transition from spillway crest to chute, and the discharge onto the river bed, have been in accordance with expectations.

Most of the tributary dams are provided with sluices at low elevations so that water may be released to provide storage for floods. Discharge occurs through the sluice at high velocity and at a relatively high rate of discharge per foot of width. Aprons designed for satisfactory operation with spillway discharge usually do not have a sufficiently deep tailwater to care for the concentrated energy of the sluice discharge. Modifications are frequently necessary for satisfactory performance.

At Norris and Hiwassee dams the aprons were long enough so that the sluice discharge could be taken care of without changing the apron. It is necessary, however, that sluices opened for discharge be arranged symmetrically on the apron whenever possible. If this precaution is not observed, large eddies form, partly on the apron and partly on the river bed below. The velocity in these eddies is high and may result in undesirable erosion of the river bed.

At Cherokee Dam it was necessary to modify the apron materially to care for the energy of the sluice discharge. A simple, level apron with a plain sill at the end was entirely satisfactory for spillway discharge. In order to dissipate the energy of sluice discharge, however, it was necessary to introduce the flow at a low elevation by depressing the bucket, to provide a series of steps ascending to the main apron elevation, to enlarge the end sill, and to extend the apron paving, adding another sill at the end of the extension. It was also necessary to modify the sluices, expanding the outlets laterally so that the discharge could be spread over the apron as much as possible. The modifications made are shown in Fig. 2. Performance of this stilling basin when the opened sluices were located symmetrically was very satisfactory.

A curious phenomenon has been noticed during the operation of the sluices at Hiwassee, Cherokee, and Douglas dams. These sluices discharge beneath the water surface at very high velocity. During their operation there is visible in the water on the apron a series of lightning-like flashes which appear to originate in the boundary between the rapidly moving jet and the quieter water on the apron, and occur irregularly but frequently at average intervals of 2 or 3 sec. These flashes are brilliant enough to be seen in sunlight, but attempts to photograph them have not been successful. They may be electrical in nature, but their cause is unknown. They are mentioned in the hope that similar occurrences may have been noticed by other observers who may have an explanation for them.

Chatuge Dam on the Hiwassee River, upstream from Hiwassee Dam, is operated solely as a storage dam since

SILT SLUICE AT OCOEE NO. 3 DAM DISCHARGING 3,000 CU FT PER SEC

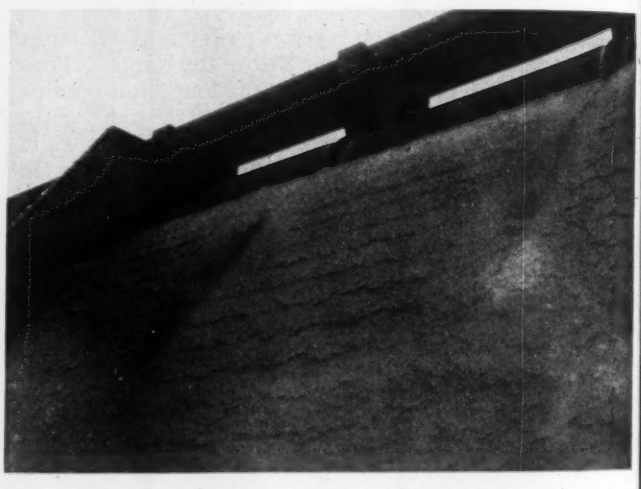
there is no power house there. Water is released to Hiwassee Dam through a low-level outlet which operates frequently and for long periods. Control of the discharge is effected by means of a Howell-Bunger valve, which discharges water in a diverging conical jet. Without control this jet would spread widely and cause undesirable erosion below the outlet structures. To avoid such erosion, the outlet valve was surrounded by a concrete structure so designed that the diverging elements of the jet were turned back upon themselves and most of the energy destroyed. Operation of the structure shows that its performance is satisfactory and that no erosion of any consequence has occurred.

Nottely Dam is provided with the same type of low-level outlet as Chatuge Dam. The dam was completed and filled before the Howell-Bunger valve was received, and it became necessary to release water without the valve. The discharge was reduced to a safe value by building a temporary bulkhead across the upper portion of the tunnel outlet, holding it in place by means of the bolts provided for mounting the Howell-Bunger valve. The emergency gate at the entrance to the tunnel was used as a control valve, being operated only in the closed or fully open position. In addition to providing this bulkhead, it was necessary to restrict the spreading of the jet by means of temporary timber bulkheads in the control structure. The jet issuing from the tunnel was thrown into the air to a height of 50 to 60 ft and struck the river bed about 175 ft downstream. It fell in the original river bed without damage to the banks. This operation was not intended to be permanent but served very well as a temporary expedient.

Ocoee No. 3 Dam was built in a region where there is an extreme amount of surface erosion, and it is expected that considerable silt will be deposited in the reservoir. Water for the turbines is supplied through a tunnel 13,000 ft long, which leaves the reservoir immediately above the dam. In order to prevent the accumulation of silt deposits at the entrance to this tunnel, two sluices have been built through the dam just in front of, and below, the tunnel entrance. Discharge from the sluices is carried into the river below the dam at a high velocity, through a channel which is built on a curve. Because of the curvature and the high velocity of flow, it was necessary to superelevate the floor of the channel in order to keep the flow from being concentrated along the outside wall. Although the model tests for this structure were made at a scale of 1:100, the performance of the prototype is quite satisfactory.

The sluices through these tributary dams have introduced a number of problems peculiar to the sluices themselves. Chief among these is the problem of cavitation damage and its prevention. Examination of the Norris sluices after the first year's operation showed that pitting of the metal lining had occurred below the gate slots and below weep holes and other irregularities of the surface. The damaged areas were repaired by grinding them smooth and by filling identations in by welding where the depth of pitting was greater than $^3/_{16}$ in. In addition, the probability of future damage by cavitation was lessened by constricting the outlets of three of the most frequently used sluices. The purpose of the constriction was to increase the pressure in the barrel of the sluice by raising the hydraulic grade line. A "Symposium on Cavitation" by the writer was published in the September 1944 issue of PROCEEDINGS.

Damage to the sluices of Madden Dam on the Chagres River in the Isthmus of Panama has occurred because of improperly shaped entrances, which caused cavitation. Reports of this damage led to an investigation of the shape of the entrances for the sluices of Hiwassee Dam.



WAVES ON THE SPILLWAY OF NORRIS DAM CAUSED BY VIBRATIONS OF THE NAPPE—DISCHARGE IS 2,000 CU FT PER SEC

Cavitation in these sluices was prevented by constricting the outlet, thus raising the hydraulic grade line and increasing the pressure, and by providing a bell-mouth entrance so that the rate of curvature did not reduce local pressures to the cavitation point. Subsequent measurements on the prototype have shown that the measures adopted were successful. No pressures below atmospheric have been found, and it is believed that there is no damage to this structure by cavitation.

Similar protective measures were adopted for the sluices of Cherokee and Douglas dams. These sluices were constricted at the outlet as well as flared to spread the discharge. Measurements of pressures in the Cherokee sluices have shown that they do not drop below atmospheric at any point, and it is believed that these sluices are also safe against cavitation.

Design of Bridges Against Wind

I. General Considerations—Aerostatic Stability

By D. B. Steinman, M. Am. Soc. C.E.
Consulting Engineer, New York, N.Y.

MODERN experiences with bridges subject to extreme wind forces have revived the interest of engineers in this subject. One who has devoted much time and study not only to the theories involved but also to the resulting treatment of the structures is Dr. Steinman. He has prepared a series of articles, nonmathematical in character. In this, the first, he mentions important failures due to wind and gives his views as to the great, but frequently overlooked, dangers from uplift. Engineers, he believes, should cooperate in study and research on this critical problem, thus drawing the profit of improved knowledge from adversity. This article was originally presented before a meeting of the Metropolitan Section of the Society.

ON December 29, 1879, a notable bridge disaster occurred—thirteen 245-ft truss spans of the Tay Bridge in Scotland were blown down during the night, with the destruction of a train and its 90 passengers. That catastrophe has been almost forgotten; it is difficult to find any reference to it in recent engineering literature. Nevertheless, the Tay Bridge disaster marked a turning point in the development of bridge engineering. It awakened engineers to the necessity of considering wind pressure in bridge design.

When opened in 1877, the Firth of Tay Bridge was considered one of the wonders of the age. It consisted of eighty-four truss spans, of wrought iron, with piers of large cast-iron cylinders erected on a base of brick and stonework. The designing and construction engineer was Sir Thomas Bouch, an eminent bridge builder. On the strength of his reputation in building this bridge, he was engaged at the time in drawing up designs for a proposed suspension bridge over the Firth of Forth.

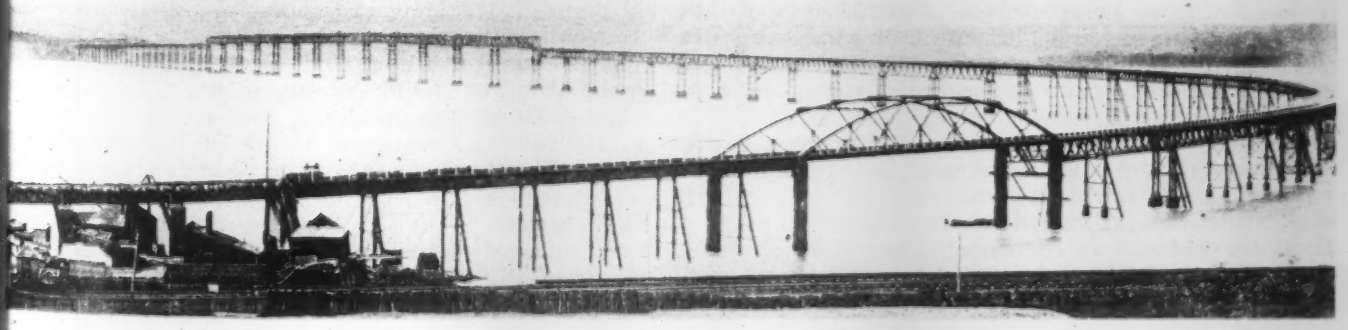
On the night of December 29, 1879, a passenger train composed of six coaches, a locomotive, and a brakeman's van crashed through the Tay Bridge, a flaming rocket, into the turbulent waters 88 ft below. The wind had reached a velocity of 72 to 80 miles an hour and apparently had blown down thirteen of the truss spans. Not a single person survived, and no one knows exactly what happened.

The design Bouch had prepared for the Forth Bridge was a most interesting suspension type, with two spans of 1,600 ft and a beautiful system of cable stays radiating upward from the ends of the spans to the panel points of the cables. When the British investigated the Tay Bridge failure, their report blamed the designing engineer for inadequate allowance for wind loads. The reputation of the builder was shattered, and his engagement to design the great Forth Bridge was canceled. The shock was too great for him, and he died a few years later.

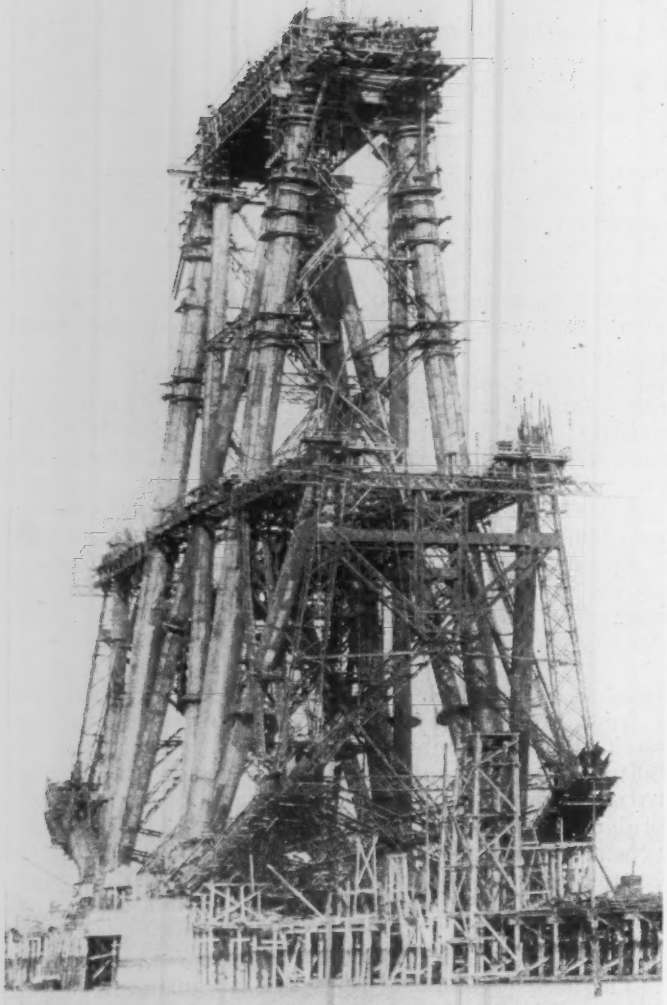
The disaster brought in its wake extensive studies of wind pressure on exposed surfaces, and also hastened the substitution of steel for wrought iron as a bridge material. The new engineers of the Forth Bridge, Fowler and Baker, conducted extensive experiments to measure the intensity of wind pressure at various velocities, and on the basis of their observations of pressure recorded on a vertical plate 10 ft square, they decided to design their monumental cantilever bridge for a wind pressure of 50 lb per sq ft, despite the tubular sections of the members. In addition, conspicuous stability against overturning was provided by the strongly battered towers and trusses, providing a broad base for a bridge otherwise narrow for its height. The cantilever type as glorified in this structure held supremacy for the ensuing forty years, and the development of the suspension type was held back for the same period.

LESSONS LEARNED FROM TAY DISASTER

The present established practice of figuring wind pressure in all bridge design, and of using steel instead of the weaker wrought iron, may both be credited as lasting beneficial consequences of the Tay Bridge catastrophe. The excess steel in the great Forth Bridge gives it a vast reserve of strength, contributing to its longevity. This gigantic structure may be regarded as a monument to the lessons learned from the failure of the smaller bridge over the Firth of Tay. Completed in 1889, with two



Tay Bridge in Scotland, High Part of Which (13 Spans near Far End) Was Lost in Disaster of December 29, 1879
From *The Forth Bridge*, by Philip Phillips, Edinburgh, 1890



A MAIN TOWER SECTION OF FORTH BRIDGE UNDER CONSTRUCTION
From *The Forth Bridge*, by Philip Phillips

main spans of 1,700 ft each, the Forth Bridge held the world's record for span length for a generation.

While the Tay Bridge disaster is a forgotten chapter in bridge engineering history, its lesson is preserved in the wind-pressure clause in bridge specifications. Engineers learned an elementary lesson, disposed of it by freezing it into standard specifications, and then gave it little further thought for sixty years until awakened by the impact of two new catastrophes. The failure of the Tacoma Narrows Bridge in November 1940 dramatically exemplified the aerodynamic effect of wind, while the Chester, Ill., Bridge failure (July 29, 1944) exemplified the aerostatic effect. The two are related; and both disasters have their lessons for the profession.

Since the Tay Bridge disaster in 1879, bridge engineers had been conscious of the importance of designing bridges to withstand static wind pressure. But their attention was directed only to the horizontal pressure of the wind. The vertical component was completely overlooked or ignored. Old handbooks, like Trautwine, briefly recorded the fact that wind had been known to lift the roofs of buildings, but bridge engineers did not think that this fact had any bearing on their work. Bridge specifications and textbooks made no mention of wind uplift. Aeronautic engineers, of course, knew the significance of the vertical component, or lift, and utilized the fact that, even at small angles of attack, this lift can amount to many times more than the horizontal component, or drag. But the bridge engineer continued to work in a separate insulated compartment of technical knowledge.

The Tacoma failure forcibly directed the attention of bridge engineers to the significance of the vertical component of wind pressure. Tests made over four years ago on models of the Tacoma section showed that the lift force, under some conditions, amounts to three or four times the horizontal force. I called attention to that significant fact in a published discussion in 1941.

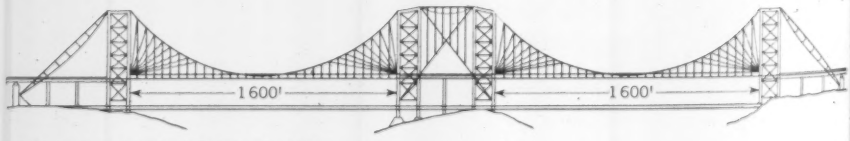
To explain the failure of the Chester Bridge it is not necessary to assume a tornado, or a wind velocity of 161 or 185 miles per hour. Those calculations ignore the effect of wind uplift. With the overturning moment due to wind uplift included, a much lower wind velocity is found sufficient to explain the failure.

A wind strong enough to uproot trees is not necessarily a cyclone or a tornado. Such winds have occurred repeatedly in almost every part of our country. On the exposed areas of bridges, including built-up sections and structural shapes, the potential wind pressure at 100 miles per hr is 50 lb per sq ft—rather than 30. Using the lower value instead of the higher corresponds to designing for an 80-mile wind instead of a 100-mile wind. That critical wind pressure may be effective over a considerable length is indicated by the Tay Bridge failure, where destructive action affected a length of one-half mile.

For the design of the Birchenough arch bridge in Rhodesia (1,080-ft span, completed in 1935), wind-tunnel tests were made on a large-scale model (1:32) of $2\frac{1}{2}$ panels of the arch truss, composed of sheet metal reproducing the actual sections of the members, with gussets, latticing, etc. These tests yielded coefficients giving for a normal horizontal wind of 100 miles per hr a normal pressure of 68 lb per sq ft of projected area. For a 100-mile horizontal wind at 30 deg from normal, the corresponding normal pressure increased to 82 lb per sq ft, with an accompanying longitudinal component of 21 lb per sq ft. On the suspended deck, a normal horizontal pressure of only 37 lb per sq ft was indicated for a 100-mile wind. Apparently no measurements of the vertical lift were taken.

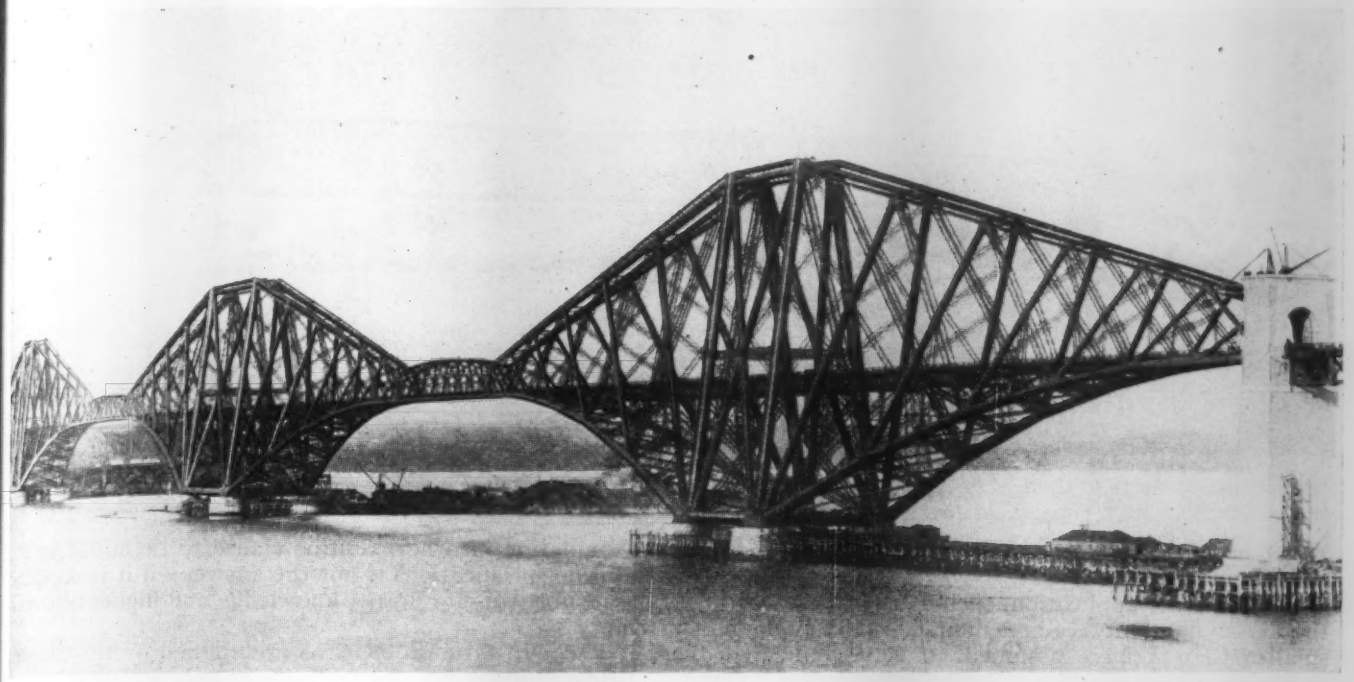
In the design of truss members and bracing, the insufficiency of the assumed wind pressure is not so serious, as these members are normally designed with the conventional factor of safety to take care of abnormal conditions. But in figuring stability against sliding or overturning, the necessary provision of a factor of safety is too easily overlooked. It is not sufficient to show that a structure is stable against sliding or overturning at the conventionally specified wind pressure. The small additional investment to provide positive anchorage against overturning or sliding by any potential wind is more than justified economically.

The Chester Bridge was narrow for its height, having only a 28.5-ft spacing for trusses 60 to 100 ft high. This



© "Engineering," London, Feb. 13, 1880

BOUCH'S SUSPENSION DESIGN FOR FIRTH OF FORTH BRIDGE



FIRTH OF FORTH BRIDGE JUST BEFORE ITS DEDICATION
From *The Forth Bridge*, by Philip Phillips

condition naturally accentuated the problem of sufficiency of anchorage against overturning.

Assuming a wind velocity of only 100 miles per hour, consider the three principal forces acting on the section per linear foot: the dead weight, 4,600 lb; the horizontal wind pressure, 1,000 lb acting at an average height of 30 ft; and the unknown vertical lift force. It is true that the horizontal force alone is insufficient to explain overturning; it would have to be 2.2 times as great, which would require a 48% increase in wind velocity. But the addition of wind uplift changes the picture, both for overturning and for sliding.

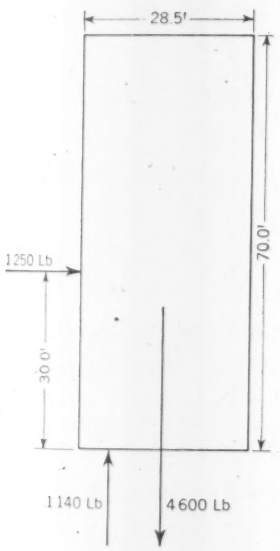


FIG. 1. FORCES ON A 1-FT VERTICAL SECTION OF CHESTER BRIDGE, INCLUDING POSSIBLE UPLIFT

Even a horizontal wind may produce vertical lift. A small inclination of the wind from the horizontal will materially augment the effect, producing a substantial lift force, at the same time augmenting the horizontal force. There is no guarantee that the wind will always be exactly normal and horizontal; and no bridge design is complete if it ignores the possibility of wind uplift.

For an accurate calculation of the overturning moments, the aerodynamic constants of the section are required. These can be easily and quickly obtained in a simple wind-tunnel test on a section model, to determine the coefficients of drag, lift, and torque. A systematic series of static wind-tunnel graphs covering the range of typical bridge sections is urgently needed by the profession. From such series, the aerodynamic characteristics of any section could be reliably estimated without requiring wind-tunnel tests on each new design. For the past several years, I have been pleading for such systematic information.

My own graphs and formulas, based on the limited experimental data I have been able to obtain, indicate that the coefficient of lift (C_L) on bridge sections may amount to as much as 1.0 to 1.5. Note that these values assume only a small angle of attack, less than 5 or 10 deg from the horizontal. These values represent an upward pressure of 25 to 38 lb per sq ft of width. For the Chester Bridge, this would mean an uplift of 760 to 1,140 lb per lin ft of span. This is equivalent to neutralizing $^1/_6$ to $^1/_4$ of the dead weight of the bridge in resisting overturning. At the same time the horizontal force (C_D) may be increased as much as 25% by the assumed angle of attack. Under these conditions, a 36% coefficient of friction would be required to prevent sliding off the pins.

Moreover, the center of pressure of the vertical uplift may have a greater lever arm than the center of gravity of the section. This is true for all aerodynamically stable sections. Under some conditions, the center of lift may be near the windward quarter-point of the section (Fig. 1). This would yield an overturning moment, per linear foot, of possibly 24,000 ft-lb from uplift, added to 37,500 ft-lb from the augmented horizontal component, or a total of 61,500 ft-lb, against a resisting moment of 66,000 ft-lb from the dead weight. The resulting factor of safety against overturning is only 1.07.

Unless effective positive anchorage is provided, overturning would result in a 104-mile wind. This is too close to the 100-mile wind at which a bridge is assumed to be safe. Furthermore, if the coefficient of sliding friction on the pins is taken as 20%, failure by sliding off the pins would result in a 75-mile wind. A design figured without considering uplift is thus revealed as potentially unsafe when uplift is included. I use the qualifying adjective "potentially" because static wind-tunnel tests on a section model would supply the requisite aerodynamic constants. Without this information, the de-

THE CHESTER, ILL., BRIDGE OVER THE MISSISSIPPI RIVER

signer has to assume the most unfavorable data within the potential range.

For a specific application, such as the Chester Bridge, the aerodynamic forces and moments may be larger or smaller than I have assumed. I have indicated the simple tests that would supply the information. Observations on existing bridges are also needed to determine the potential range of angles of attack of the wind (above or below the horizontal) to be considered in design. Tests are also needed to determine the length over which critical wind pressure may act.

The new knowledge may not necessitate any modification of our present specifications and design procedure for the calculation of wind stresses in truss members and bracing. But it will supply needed data for the scientific design of anchorage details to prevent sliding, uplift, and overturning. Until these gaps in aerodynamic knowledge as applied to bridges are filled, it is incumbent upon the designer to recognize the potentialities and to play safe. The cost of ample anchorage is relatively trifling.

Some engineers are impatient with scientific analysis and research and are in a hurry to see the final answer recorded and frozen into a routinized specification that can be mechanically applied. That sort of thing discourages creative originality, continuing research, and true progress, and encourages unthinking application and professional sterility. The engineer who regards all new knowledge as esoteric, something for so-called "experts" to hand him on a silver platter, will make no real contributions to the progress of the profession. The bridge engineer of today can no longer regard aerodynamics as a foreign field; he needs a better understanding and working knowledge than merely to have someone tell him whether to use 30 or 50 lb per sq ft.

The practice of attempting to cover all design requirements in cut-and-dried specifications, and of using such specifications as a sufficient criterion of good design, may be satisfactory for routine, repetitious designs of ordinary character—simple types, short spans, and normal proportions. But when an engineer undertakes projects of specialized types, beyond the beaten track, a higher obligation is imposed upon him. We must not make a fetish of specifications. They must be supplemented by an inquiring mind and by good judgment.

Engineering failures are the price we pay for progress. "The only man who never stumbles is the man who stands still." But when a failure occurs, let us make sure that we learn all we can from the experience, to make it a real stepping stone to enriched understanding. The mere formulation of a new routine clause to be added to the routine specifications is not the answer—if it is accepted as a substitute for deeper knowledge and higher responsibility.

The aerostatic problem, as exemplified by the Chester Bridge failure, is comparatively simple and elementary. The aerodynamic problem, as exemplified by the Tacoma Bridge fai ure and by the oscillations of other suspension bridges, is one of much greater difficulty and complexity. (It will be discussed in subsequent articles of this series.) It virtually requires the creation of a new science combining the essentials of three different fields of specialized knowledge—the deflection theory of suspension bridges,

WRECKAGE OF THE CHESTER BRIDGE, FROM ILLINOIS SIDE

the science of aerodynamics, and vibration analysis. Even existing knowledge in aerodynamics has proved inadequate, necessitating new research and analysis.

An engineering failure is an indictment, not of an individual but of the entire profession. If we profit from the experience through the stimulation of new creative thinking and constructive research, and through the broadening and enriching of our professional equipment, these failures will not have been in vain.

Army Sewerage in Hawaii

Improvisation Has Paid Dividends in Maintaining Health

By BELDEN S. TUCKER

CHIEF, SANITATION UNIT, CONSTRUCTION SERVICE, CENTRAL PACIFIC BASE COMMAND, U.S. ARMY

THERE are eight principal islands comprising the Hawaiian chain—Hawaii, Maui, Oahu, Kauai, Molokai, Lanai, Niihau, and Kahoolawe. Their combined area is 6,454 sq miles. The capital city of Honolulu is located on the third largest island, Oahu. According to the earliest records, the first migration of Polynesians to the islands occurred about the sixth century, A.D. After the thirteenth century, voyages ceased and all communication with the islands was lost. In 1778 they were rediscovered by Captain John Cook and their beauty and importance made universally known.

Until 1874, the islands were ruled by Kings Kamehameha I, II, III, IV, V, and their descendants. King Kalaukaua, followed by his sister Queen Liliuokalani, ruled from 1874 until 1893, when the monarchy was overthrown and a republic established. In 1898, the islands were annexed to the United States, and in 1900 they were organized as a territory—their present form of government. Their location in the Pacific makes them exceptionally important, from both military and commercial standpoints, a fact which was well emphasized by the Japanese in their action of December 7, 1941, although their dreams of conquest were exceptionally optimistic.

It was not until 1878 that the importance of improving the sanitary conditions of the islands, particularly in the city of Honolulu, was realized, as evidenced by the issuing of sanitary instructions to the people in both English and Hawaiian. The instructions contained definite regulations for the construction and maintenance of pit-type privies, which were the only means provided for the disposal of fecal matter. The necessity for a water-carriage sewerage system was realized as early as 1880, and many recommendations were made periodically to the legislative assemblies for such a system. However, it was not until the cholera epidemic of 1885 that definite action was taken toward the construction of a suitable sewerage system. The work was initiated in 1889 and was completed in 1911, with 58 miles of sewer and 2,600 service connections. By 1911, however, parts of the system were already overloaded, and the construction of intercepting sewers was initiated. Several of the Army posts within the city of Honolulu are at the present time served by this sewerage system.

EXPANSION FOLLOWING JAPANESE ATTACK

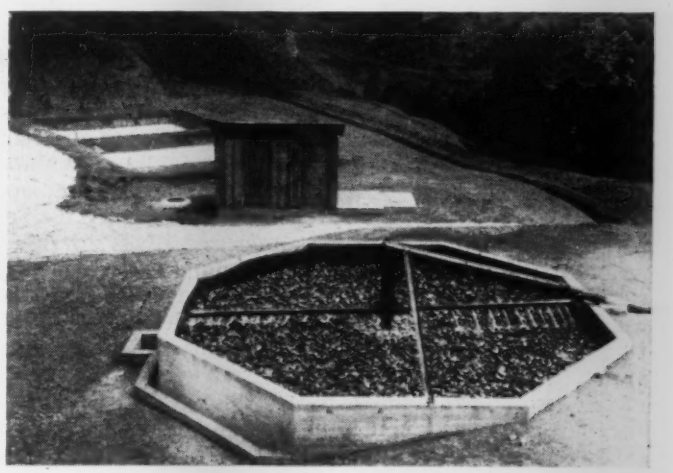
Before December 7, 1941, Army sewerage systems on established posts in the Hawaiian Islands were generally adequate, although some minor expansions were undertaken. Sewerage systems for several new posts were designed and completed earlier in 1941, but it was not until directly after the Japanese attack that the Army sewerage business in Hawaii really began to boom. Established posts were expanded beyond previous conceptions, which required major rehabilitation of many of the sewerage systems. New camps, ranging from 100 to 15,000 men, were designed and constructed almost overnight.

The variety of sewerage problems was almost infinite, and the terrain of the islands is not particularly suited to simple solutions. Areas favorable for development near the ocean are generally level and near sea level, requiring frequent pumping and construction in ground water, as well as provisions for protection of the beaches from contamination. Inland areas, on the other hand, confront the designer with rugged terrain, steep gradients, deep canyons and gullies, and few streams suitable for use as receiving bodies for sewage. In addition the subsurface structure is extremely variable, the islands being of volcanic origin. Bearing conditions and construction difficulties change greatly with different localities and even within the area occupied by the smallest installation. Cesspool construction is obviously an art, and much has been learned from the local old timers who have had considerable and valuable experience in this still thriving business. A number of the islands' cities and villages are still without sewerage systems.

TREATMENT PROCESSES

Army camps constructed after December 7, 1941, were of two general types—semi-permanent, or those provided with the usual water carriage facilities; and temporary, or those utilizing pit or vault-type latrines and separate shower and washing facilities. In the latter type of camp, the liquid waste has a somewhat higher biochemical oxygen demand (B.O.D.) than the sewage from the semi-permanent type of camp, but does not require the same degree of treatment to stabilize it, or to make it suitable for discharge into a receiving body. The maximum degree of treatment installed for camps of the temporary type is primary settling and chlorination.

Whether this or a lesser degree of treatment is required, is of course dependent entirely on the use and capacity of the receiving body. Treatment of the sewage from camps of the semi-permanent type is accomplished by the usual procedures to the degree necessary for proper stabilization. In many instances it has been possible to discharge sewage from these camps directly into the ocean without treatment. In camps of both



HIGH-RATE TRICKLING FILTER INSTALLED AT A HAWAII POST
Chlorination Contact Tank and Sludge Beds in Background

types, grease traps are provided for all mess halls. Many different designs are used, depending on the materials available at the time of construction.

Pit latrines constructed for camps of the temporary type are of considerably greater capacity than those generally advocated. One cubic foot per capita per month is used as the design basis for volume. The pits are made large enough for nine months' continuous use. The average pit is 15 ft deep and extends for the full length and breadth of the latrine superstructure. Fly breeding and odors are controlled by spraying the contents with oil or with compounds developed for such purposes by the Army Medical Corps. The design figures used have proved satisfactory, as shown by time-usage checks on typical latrines.

Where the ground-water table is close to the surface, or where rock excavation is encountered, pit latrines become impractical and the vault type is used. This type is similar to the pit type, except that the pit is constructed of watertight concrete. At each toilet seat agitators are provided, consisting of a rod which extends for the full depth of the pit. Flat plates are welded to the rod in the lower section so that the sludge is stirred when the agitator handle is moved up and down. The contents of these pits are pumped out with diaphragm sludge pumps when the maximum sludge level is reached. Sludge is then disposed of by burial in approved localities. The vault is primed with water and pine oil disinfectant when initially placed in use. Fly breeding is controlled when required by suitable chemicals.

"VICTORY"-TYPE LATRINES

Latrines suitable for water-carried sewage are provided for semipermanent and permanent camps. These latrines are of two types, those having the usual ceramic sanitary fixtures and a modified or so-called "victory type" utilizing a concrete toilet pit, which is flushed out with stored shower and wash-basin waste water. Lavatories for the "victory-type" latrines consist of wash troughs or basins lined with galvanized iron. The concrete toilet pit replaces the ceramic type of water closet. Urinals are in the form of troughs with perforated pipe for clearing with water. The urinals are equipped with treadles or flush tanks to effect a further saving in water. Nearly a hundred of these "victory-type" latrines have been built and have effected large savings in critical materials and water usage.

Enlargement of sewerage systems at established posts has been accomplished in the usual manner by constructing intercepting sewers and by increasing the capacities of pumping stations and sewage treatment plants. At one post the tremendous overload on the sewage treatment plant was relieved by intercepting the sewage from a large tributary area and providing a new treatment plant in a suitable location, but in a different drainage area. In another instance, the 600 trickling-filter nozzles of a treatment plant required immediate replacement, and an additional 200 nozzles were needed for new filters, but commercially manufactured brass nozzles of standard design could not be obtained. Of necessity, improvised nozzles made from pipe caps, and a small amount of wire and babbit metal, were designed and fabricated locally and installed. They provide much better spray distribution and freedom from clogging than the original standard nozzles.

Small, semipermanent camps are provided with septic tanks and cesspools or drilled wells for treatment and disposal where possible. Similar camps in areas where disposal by such means is impractical are provided with complete treatment consisting of a septic tank, filter, and chlorinator. The effluent is discharged into a nearby gulch or stream. The method of treating waste from camps of the temporary type has already been mentioned.

CONTACT FILTER OF SPECIAL DESIGN
Note Rock Submergence Level

FORTY TREATMENT PLANTS

Since 1941, forty treatment plants have been constructed, ranging in capacity from 300 to 20,000 men, and over 100 sewerage systems for smaller populations. The major plants employ various combinations of the following treatment units: Imhoff tanks, plain sedimentation tanks, septic tanks, low-rate and high-rate trickling filters, lath filters, intermittent sand filters, contact filters of special design, final sedimentation tanks, chlorination contact tanks, sludge digesters, and sludge beds. One plant has been constructed for the treatment of laundry waste; this plant utilizes a modification of the activated sludge principle to obtain the required degree of waste stabilization.

There is little novel or new in the basic design of these units except for the contact and high-rate trickling filters. An accompanying photograph shows a typical high-rate trickling filter. This type is designed to effect a saving in critical materials, to utilize materials locally available, and to meet high priority construction demands which could not be met within the construction time deadlines if commercially manufactured equipment obtained from the mainland were used. These filters are designed for a B.O.D. loading of 2.0 lb per cu yd of rock and a surface loading of 9 mgd. The center bearing detail of the distributor is greatly simplified by feeding the filter from above rather than from below, which is the usual practice. These filters have more than met expectations both in cost and in results. The average B.O.D. reduction provided is 60%.

The contact filters used were designed and constructed for the same reasons as the high-rate trickling filters. The former are much less difficult to construct than the latter because of the simplified forming and because the distributor unit is not required. The contact filter is composed of two beds of rock and a discharge siphon for each. Each bed is used every other day to provide necessary resting periods. The filters are designed for two hours' contact at average flow and are surprisingly free from odor and fly nuisance. The B.O.D. reduction provided by this type of filter averages 40%. Only a few contact filters have been constructed and these for populations of 1,000 or less. However, they have proved indispensable in meeting high priority emergencies. A filter of this type is shown in an accompanying photograph. Table I gives average B.O.D. reductions derived

from several of the representative plants for which chemical analyses of sewage are available.

This résumé of Army sewerage in Hawaii is necessarily very general. Data of particular interest, such as relative locations of treatment plants, distribution among the various islands, and population loadings, cannot be released at this time. Such data will, however, be available at a later date and may provide valuable additions to sewerage knowledge and experience, and have a marked effect on the general development of sewerage in the Hawaiian Islands.

Sewerage problems have been many, the greatest being the procurement of construction materials and equipment. Improvisation has been at a premium and has paid high dividends in maintaining the health of the men who are winning the war for us. Even at this late date, pumps and special equipment are urgently required from the mainland to replace such equipment salvaged from dumps and junk yards and hammered into usable shape to permit immediate utilization of important facilities.

The cooperation afforded by the U.S. Army Medical Corps, the U.S. Navy, the Territorial Bureau of Sanitation, the City and County of Honolulu Department of Public Works, local dealers, and many of the island plantations, has made it possible to meet construction deadlines and to maintain the present high standard of Army, Navy, and civilian health in the islands. The design and construction discussed herein is a function of the Construction Service, Central Pacific Base Command, U.S. Army.

TABLE I. AVERAGE OVERALL B.O.D. REDUCTION FOR TYPICAL ARMY TREATMENT PLANTS IN HAWAII

PLANT DESCRIPTION	AVG. B.O.D. REDUCTION	NUMBER OF PLANTS IN AVG.	TYPE OF CAMP
ST, CF, CL	64%	4	Semi-permanent
IT, LRF, CL	91%	3	Semi-permanent
ST, LRF, CL	86%	1	Semi-permanent
ST, HRF, CL	75%	2	Semi-permanent
SDT, CL	76%	4	Temporary
ST, ISF, CL	96%	1	Semi-permanent

Legend:

CF	= contact filter	IT	= Imhoff tank
CL	= chlorination	LRF	= low-rate trickling filter
HRF	= high-rate trickling filter	SDT	= sedimentation tank
ISF	= intermittent sand filter	ST	= septic tank

Engineers' Notebook

Suggestions and Practical Data Useful in the Solution of a Variety of Engineering Problems

Unsymmetrical Bending and S-Polygons

By W. L. BLANKENBURG, JUN. AM. SOC. C.E.
STRUCTURAL DESIGNER, THE H. K. FERGUSON COMPANY, CLEVELAND, OHIO

FOR a long time engineering textbooks have included material on the use of an S-polygon for handling cases of unsymmetrical bending. If the plane of bending moment does not coincide with one of the principal axes, then the usual formula, $f = Mc/I$, does not apply. A very complete treatment of the subject of unsymmetrical bending was given by Prof. L. J. Johnson, M. Am. Soc. C. E. (TRANSACTIONS, Am. Soc. C.E., Vol. 56, 1906). In the following article, a brief method is given which substitutes equations based on analytical geometry for the graphical solution developed by others.

For any point u, the stress produced by unsymmetrical bending is

$$f_u = \pm \frac{My \sin \theta}{I_x} \pm \frac{Mx \cos \theta}{I_y}$$

in which M is the bending moment in the plane of bending moment. The stress at point B is

$$f_B = \frac{-M}{\dfrac{I_x I_y}{I_y y_B \sin \theta + I_x x_B \cos \theta}}$$

This can be shortened to $f_B = -\dfrac{M}{S_B}$. Similarly the stress at point D is

$$f_D = \frac{+M}{\dfrac{I_x I_y}{I_y y_D \sin \theta + I_x x_D \cos \theta}}$$

This can be reduced to $f_D = +\dfrac{M}{S_D}$. Both S_B and S_D can be determined graphically. They are to same scale as e and f, etc., and the construction is obvious from Fig. 1.

The equation of the line joining e and f is $x/f + y/e = 1$, and the equation of the line on which S_B is found is $y = x \tan \theta$. These equations can be solved for x. Then the distance S_B equals the value of x previously

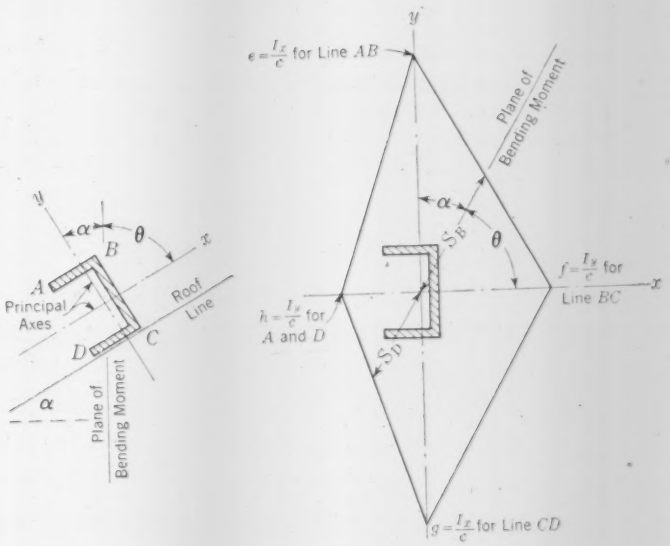


FIG. 1. CONSTRUCTION USED TO SOLVE FOR STRESS PRODUCED BY UNSYMMETRICAL BENDING

found, multiplied by sec θ. The expression for S_B reduces to the following equation:

$$S_B = \frac{ef \sec \theta}{f \tan \theta + e} = \frac{ef \csc \alpha}{f \cot \alpha + e}$$

To solve for S_D, use the same values for α and θ. The g is substituted for e, and h is used in place of f. The expression for S_D then takes the following form:

$$S_D = \frac{gh \sec \theta}{h \tan \theta + g} = \frac{gh \csc \alpha}{h \cot \alpha + g}$$

If the cross section of the beam has reentrant corners, as in the case of most rolled steel sections, the S-polygon is drawn for the corners that lie on the polygon that envelopes the cross section without forming any reentrant corners. In other words, only the outside corners need to be investigated unless it is a beam of two materials.

Wheel-Load Distribution to Timber Stringers

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HEAD BRIDGE DESIGNER, LOUISIANA DEPARTMENT OF HIGHWAYS, BATON ROUGE, LA.

IT has been recognized for some time that the prevailing specifications regarding wheel-load distribution to timber stringers are very inadequate, especially as regards horizontal shearing stress in the stringers. As the demand for timber trestles and bridges designed for heavy truck loadings has become more pressing, it has become more and more difficult to meet the requirements of existing specifications (American Association of State Highway Officials, 1941 edition), since the stringers usually obtainable are in small sizes, that is, not over 6 by 16 in.

It appears that the A.A.S.H.O. specifications for computing horizontal shear in rectangular timber beams were obtained by following certain assumptions. It was assumed that the wheel load would be distributed to stringers in direct proportion to the distance from the center of the flooring span. This reasoning is fallacious. Actually the distribution varies, not lineally, but inversely as the square root of the product of the distances from each support to the point of application of the load. This statement is valid only when the deflection of the stringers is due only to flexure.

The problem of load distribution to flexible and closely spaced timber stringers may be likened to the problem of the bending of a bar infinitely long, resting on an elastic foundation, for which solutions for various types of loadings are already available. For the case of timber floors resting on timber stringers, and supporting wheel loads, the floor is regarded as the flexible bar, the stringers as the elastic foundation, and the wheel loads as concentrated loads.

The solution for this case, when the deflection of the stringers due to flexure only is considered, yields the following equation:

$$Q = \frac{PS}{2} \beta \phi_{\beta x} \qquad (1)$$

where

$\phi_{\beta x} = e^{-\beta x}(\cos \beta x + \sin \beta x)$
$e = 2.71828$
$\beta = \frac{1}{L} \sqrt[4]{\frac{3E_s I_s L}{4E_p I_p S n^2 (1-n)^2}}$
Q = wheel load distributed to stringer under consideration
P = largest wheel load
S = spacing of stringers
L = span of stringers
E_s, I_s = modulus of elasticity and moment of inertia of stringers
E_p, I_p = modulus of elasticity and moment of inertia of flooring
n = ratio of distance that load is from support to span of stringers
x = distance from load to stringer

All units are in pounds and feet.

The function $\phi_{\beta x}$ may be obtained readily from Fig. 1.

From Eq. 1, it is readily seen that when $x = 0$, $\phi_{\beta x} = 1.0$, and the value of Q varies along the beam according to $\frac{1}{[n(1-n)]^{1/2}}$, as previously mentioned.

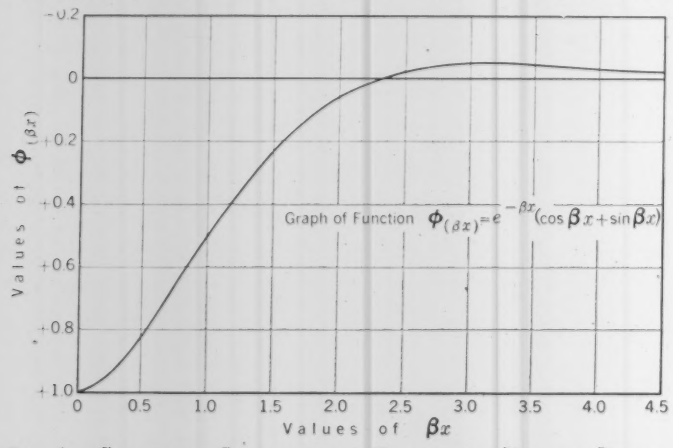


FIG. 1. GRAPHICAL SOLUTION FOR FACTOR $Q_{\beta x}$ USED IN DETERMINING DISTRIBUTION OF LOADS TO STRINGERS

EXAMPLE TO ILLUSTRATE METHOD

The following example will show the simple application of Eq. 1. A timber trestle of span $L = 19$ ft 0 in. has 6 by 16-in. stringers on 2-ft 0-in. centers, and a 4 by 12-in. plank floor. Find the wheel-load distribution to the stringers, (1) at the center of the span, and (2) at a distance from the support equal to three times the depth of the stringer, or at the quarter point, whichever is the shorter distance. The roadway width is 21 ft 0 in., and accommodates two lanes of H-10 trucks, each occupying a 10-ft 0-in. lane.

1. For a load at the center of the span, computation gives $\beta = 0.425$.

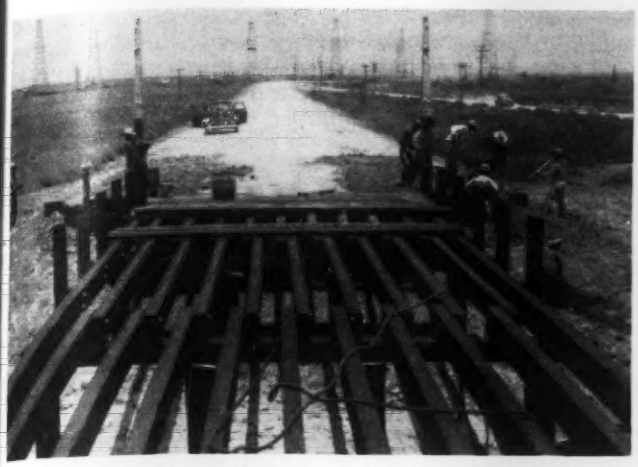
For the stringer under the load, $x = 0$: $\beta x = 0$ and $\phi_{\beta x} = 1.0$. Therefore $Q_{x=0} = 0.425P$.

Table I gives the stringer reactions when the trucks are passing each other.

The total wheel-load distribution to the stringer under consideration is

$$Q = (0.425 + 0.068 - 0.009 - 0.0085)P = 0.4755P$$

If $Q_{x=6}$ and $Q_{x=10}$ are considered negligible, then Q equals $0.493P$, which is sufficient. By the A.A.S.H.O. Speci-



AN ALL-TIMBER HIGHWAY BRIDGE UNDER CONSTRUCTION

fications, 1941 edition, we obtain for wheel-load distribution to the stringer, for two or more traffic lanes,

$$Q = \frac{PS}{3.75} = \frac{2.0P}{3.75} = 0.533P$$

2. For the load at $\frac{3 \times 15.5 \text{ in.}}{12 \text{ in.}} = 3.87$ ft from the support, computation gives $\beta = 0.526$.

TABLE I. STRINGER REACTIONS WITH LOAD AT CENTER OF SPAN

DISTANCE OF LOAD FROM STRINGER, x	VALUE OF βx	VALUE OF $\phi(\beta x)$	VALUE OF STRINGER REACTION
0.0	0.0	1.0	$Q_{x=0} = 0.425P$
4.0	1.70	0.16	$Q_{x=4} = 0.068P$
6.0	2.55	−0.021	$Q_{x=6} = -0.009P$
10.0	4.25	−0.02	$Q_{x=10} = -0.0085P$

Table II gives the stringer reactions for passing trucks, neglecting values of Q for $x = 6.0$ ft. The total wheel-load distribution to the stringer under consideration is

$$Q = (0.526 + 0.023)P = 0.549P$$

Using A.A.S.H.O. specifications, the wheel-load distribution to the stringer for two or more traffic lanes is found to be $Q = 0.767P$, which is 39% more than the value found by Eq. 1.

TABLE II. STRINGER REACTIONS, WITH LOAD 3.87 FT FROM SUPPORT

DISTANCE OF LOAD FROM STRINGER, x	VALUE OF βx	VALUE OF $\phi(\beta x)$	VALUE OF STRINGER REACTION
0.0	0.0	1.0	$Q_{x=0} = 0.526P$
4.0	2.1	0.044	$Q_{x=4} = 0.023P$

Extensive studies of various spans designed for various truck loads, for one, two or more traffic lanes, and various floors and spacing of stringers, have been conducted. The studies showed that:

1. For one, two, or more lanes of traffic, and for the ordinary lengths of timber trestle spans, the distribution of a load is so slightly improved when a second wheel comes on the stringer that it is satisfactory to consider only the largest wheel on that stringer.
2. For two or more traffic lanes, the wheel-load distribution of a dual-tire wheel applied as two half-wheel load concentrations is approximately the same as that of a dual-tire wheel applied as one wheel-load concentration.
3. For one traffic lane, the consideration of a dual-tire wheel, applied as two concentrated loads, yields a better load distribution (about 8%) than would be obtained if one concentration were used.

TABLE III. WHEEL-LOAD DISTRIBUTION COEFFICIENTS FOR TWO OR MORE TRAFFIC LANES—H 15 LOADING

TYPE OF FLOOR	STRINGER SPAN L, IN FT	SIZE OF STRINGER	STRINGER SPACING S, IN FT	COEFFICIENTS: FOR MOMENT, Eq. 1	FOR MOMENT, A.A.S.H.O. 1941	FOR SHEAR, Eq. 1	FOR SHEAR, A.A.S.H.O. 1941
Planking, 4 × 12-In.	13	4 × 16	1.58	0.44	0.42	0.48	0.71
		4 × 14	1.00	0.29	0.27	0.32	0.64
		3 × 12	0.667	0.17	0.18	0.20	0.59
	19	6 × 16	1.58	0.40	0.42	0.44	0.71
		4 × 16	1.00	0.29	0.27	0.28	0.64
		4 × 14	0.667	0.17	0.18	0.20	0.59
	25	6 × 18	1.58	0.38	0.42	0.42	0.71
		6 × 16	1.00	0.29	0.27	0.28	0.64
		4 × 16	0.667	0.17	0.18	0.19	0.59
Laminated, 4-In.	13	6 × 14	2.00	0.52	0.50	0.56	0.75
		4 × 12	1.00	0.26	0.25	0.28	0.63
		4 × 10	0.667	0.17	0.17	0.18	0.59
	19	8 × 16	2.00	0.49	0.50	0.56	0.75
		4 × 14	1.00	0.23	0.25	0.27	0.63
		4 × 14	0.667	0.16	0.17	0.18	0.59
	25	8 × 18	2.00	0.46	0.50	0.53	0.75
		4 × 16	1.00	0.23	0.25	0.27	0.63
		4 × 16	0.667	0.16	0.17	0.18	0.59
Laminated, 6-In.	13	6 × 18	3.00	0.70	0.71	0.71	0.86
		6 × 14	2.00	0.46	0.47	0.48	0.74
		4 × 10	0.667	0.16	0.16	0.15	0.58
	19	8 × 18	3.00	0.68	0.71	0.70	0.86
		6 × 16	2.00	0.46	0.47	0.46	0.74
		4 × 14	0.667	0.15	0.16	0.16	0.58
	25	10 × 18	3.00	0.66	0.71	0.69	0.86
		8 × 18	2.00	0.44	0.47	0.46	0.74
		4 × 16	0.667	0.14	0.16	0.16	0.58

Table III affords a comparison of the wheel-load distribution coefficients obtained by Eq. 1 and by the A.A.S.H.O. specifications, and is excerpted from one of the studies conducted for and by the Subcommittee on Bridges and Structures of the American Association of State Highway Officials. This subcommittee was composed of the following bridge engineers: N. B. Garver, M. Am. Soc. C.E., chairman (Arkansas); J. P. Exum, Assoc. M. Am. Soc. C.E. (Texas); N. E. Lant, M. Am. Soc. C.E. (Louisiana); and Homer X. White (Oklahoma).

Design of Sewers or Drains for Greatest Economy

By W. E. HOWLAND, ASSOC. M. AM. SOC. C.E.

PROFESSOR OF SANITARY ENGINEERING, PURDUE UNIVERSITY, LAFAYETTE, IND.

IN order to present the following rule for the economic design of sewers, it is necessary to define a certain important ratio (here called r) which appears in this as in several other similar rules for the economic design of hydraulic structures. It is the ratio of the change in cost per unit length of the sewer to the corresponding change of its slope for a given capacity.

To illustrate, let it be supposed that a particular sewer is to carry 10 cu ft per sec. According to a sewer-capacity diagram which happens to be at hand, it appears that a 27-in. sewer would need a slope of 0.00142 to carry this flow, while a 30-in. sewer would need a slope of only 0.00082 to carry the same flow. Now if the difference in cost per foot of these two sewers is 65 cents, then the value of r for this portion of the chart equals $65 \div (0.00142 - 0.00082) = 110{,}000$ cents per ft. That is to say, the change in cost per foot of sewer, divided by the corresponding change in slope for this fixed flow of 10 cu

ft per sec, gives this amount. Thus it would be easy to compute the value of r for any portion of a standard sewer design chart, providing the difference in cost per foot of adjacent sewers of different sizes were known and recorded on the chart. (It would be convenient to construct on the chart the lines of equal value of r to avoid the individual calculations just illustrated, but such lines would be limited in application to a particular set of cost data.)

Using this definition of r, the rule for economic combination of sizes and slopes of the sewers which come together at a given point becomes the following:

> Half the cost per foot of depth of all sewer sections immediately adjacent to this point, plus r for the downstream adjacent section, shall equal the sum of the r's for the upstream adjacent section.

It is assumed that the sewers all have substantially the same invert elevation at this junction. If any join at a much higher elevation, they are disregarded in the application of the rule to the others.

By half the cost per foot of depth of a given sewer section, is meant simply one half the product of the width of the sewer trench, the length of the sewer section (that is, distance to the next change in size or grade), and the cost per cubic foot of excavation. Allowance for the cost of sheeting per foot of depth can easily be made if desired.

The rule of economy just stated would of course have to be abandoned in certain instances where limitations of depth or velocity, or other special requirements peculiar to the design, might have to take precedence over economy. This rule may be used for testing the economy of design of any single group of sewer sections joining at a point or it may be used to select a balanced design for the next downstream section below the junction in making the original sewer design. It is possible to devise other rules for the economic design, or for testing the economy of design, of large portions of the sewer system by combining equations that result from the application of the simple rule to a large number of adjacent sewer junctions.

In order to illustrate the use of the rule, it will be applied to a very simple situation—the design of a single section of sewer which starts from a point of fixed elevation, and extends a fixed length to the vertical wall forming the bank of a stream. The problem is to choose the slope and corresponding size of the sewer so as to involve the least cost of pipe and excavation. Since there is no sewer downstream from the outlet, the rule becomes simply: Half the cost per foot of depth of excavation of the sewer should equal r for that sewer.

Suppose that the sewer is to carry 10 cu ft per sec and is 500 ft long, and that the trench is 5 ft wide, also that excavation costs 3 cents per cu ft. According to the rule, the value of r should then be close to

$$(500 \times 5 \times 3) \div 2 = 3{,}750 \text{ cents per ft}$$

When the flow is 10 cu ft per sec, I find from cost data on my own design chart that r between 21 in. and 18 in. is $44 \div (0.013 - 0.0057) = 6{,}000$ cents per ft; between 18 in. and 15 in. it is $37 \div (0.036 - 0.013) = 1{,}600$ cents per ft; consequently I would use the 18-in. sewer, as having a value of r somewhere between these values. It comes as close as possible to the desired value of 3,750.

The complete derivation of the rule here stated, as well as several elaborations and extensions of it, are contained in a monograph by the writer on file in the Engineering Societies Library, 29 West 39th Street, New York 18. A mimeographed copy will be supplied by the writer on request.

Our Readers Say—

In Comment on Papers, Society Affairs, and Related Professional Interests

In Favor of the Metric System

DEAR SIR: In a letter published in the July issue, Leonard C. Jordan offers objection to a change to the metric system of measurement. However, he does not advance a single logical argument against the suggested change.

In his first paragraph, Mr. Jordan makes the amazing deduction that advocates of the metric system recommend that its use be made compulsory. To my knowledge, no specific plan has been outlined for achieving universal adoption of the system. A starting point might be an agreement among engineering societies to use it exclusively in publications under their jurisdiction. Government agencies might be induced to do likewise. Its general adoption by industry would be more difficult. At any rate, compulsion does not appear to me to even enter into the question.

Mr. Jordan follows the above deduction with the statement that the established system (English system) is "now preferred by those who use dimensions and do the mathematical work connected with them." I wonder where Mr. Jordan has been living during the past ten years. During that period, at least, I have heard countless engineers and other professionals lament the clumsiness of the English system, and Mr. Jordan is the first one I have run across who prefers it. I don't doubt that there are others who prefer the retention of the English system, but if they are in a majority or even a sizable minority, they have remained strangely inarticulate.

In his second paragraph, Mr. Jordan indicates that he does have an inkling of the principal incentive for universal adoption of the metric system—the advantages, from the standpoint of international trade, of a common system of measurement. He states however, that "the United States and Great Britain have done more with feet and inches than the rest of the world has done with meters," and for some mysterious reason proposes this debatable claim as evidence that the rest of the world could change to the English system more easily than we could change to the metric system. His boast about the progress of the United States and Great Britain is patriotic, but does he expect us to believe that that progress is attributable to the use of feet and inches? The metric system is already in considerable use in both the United States and England, and is more familiar to us than is the English system to most others. That the metric system is simpler is too evident for discussion.

True, the English system could be converted into a decimal system to a greater extent than the present tenth-of-a-foot usage in surveying, but would such an abortion be more practical than the adoption of a ready-made system already in use in most parts of the civilized world? Mr. Jordan seems concerned chiefly with linear dimensions. Does he contemplate setting up weight, area, and volume systems, including tenths of pounds, tenths of gallons, tenths of cubic feet, and others?

He informs us that the metric system is "no cure for all ills" and "no guarantee against mistakes." I have never read any claims, by proponents of the system, that it is, and I cannot conceive of any system that would offer such a Utopia. Mr. Jordan suggests that the metric system involves increased opportunity for errors in locating decimal points. Perhaps he considers fractions not susceptible to error. The fact that the metric system eliminates the

use of fractions is a leading advantage of the system, and good engineers do not get gray determining the location of their decimal points.

In his last paragraph Mr. Jordan states that the period ahead will be too "trying and uncertain" to permit the disturbance of present standards. The future is always uncertain and is always trying to defeatists, and a meritorious change should not have to await the pleasure of "those who do the work that would be most seriously disrupted by the change." As has already been pointed out by many, the retooling necessary in the reconversion of war plants to civilian production offers an unusual opportunity for change-over to the metric system with minimum cost and confusion.

MAURICE N. AMSTER, Assoc. M. Am. Soc. C.E.
Chattanooga, Tenn.

Cutting and Moving Great Blocks of Stone an Ancient Art

DEAR SIR: In the September issue of CIVIL ENGINEERING, Professor Rathbun gave an interesting description of Syrian ruins, entitled "An Unsolved Problem." He has been unable to determine how the large blocks of stone, shown in his illustrations, were transported. The same unsolvable problem exists in other ruins in other parts of the world, particularly in the remains of Inca construction in Peru—near Cuzco.

I can make no suggestion that will help in the solution, except to invite attention to the fact that the art of cutting and moving great blocks of stone was known thousands of years ago and in many parts of the world. The quarrying and transporting of the Egyptian obelisks must have presented great difficulties.

I venture to invite attention to another unsolved problem, which is, why do engineers have difficulty with poetical meter? You will find false quantities in almost every engineering poetical effusion, but certainly the classics should be familiar. Lord Byron, I am sure, would be despondent to know that his name is no longer remembered and that his writings are relegated to the classification of an "old school-day poem."

HENRY WELLES DURHAM, M. Am. Soc. C.E.
New York, N.Y.

[*Editor's Note:* The shortcomings mentioned are ours, not Professor Rathbun's.]

Inconsistencies of Present System of Weights and Measures

DEAR SIR: As a general rule, the writer is not addicted to writing "letters to the editor." However, in the July issue, there is published a letter by Leonard C. Jordan, on the inadvisability of having the United States and Great Britain change to the metric system. That letter got my fingers itching.

Personally I do not care whether the United States and Great Britain adopt the metric system or not. Having studied in the United States, I am quite familiar with the anachronistic beauties of the English system, and have no ax to grind in this regard. Mr. Jordan's letter, however, does not appear to be a model of logic.

In fact Mr. Jordan says that "since Great Britain and the United States have done more with feet and inches than the rest of the world has done with meters, the rest of the world could change to feet and inches more easily than we could change in the opposite direction." I take no exception to Mr. Jordan's statement regarding the relative output of the United States and Great Britain as compared with that of the rest of the world from the time of its creation. He is undoubtedly correct, no matter what the field of endeavor. But this in spite of, rather than because of, their system of weights and measures. I hope I am entitled to wonder how much more the United States and Great Britain could have done if not saddled with their unwieldy system.

I believe Mr. Jordan will concede that we are indebted to the ancient Greeks for much of our mathematical learning. (Most assuredly this is the case only because there was no United States or Great Britain in those days; but no matter.) Their results were arrived at by the use of cumbersome old methods of mathematical notation. It does not follow, however, that simply because Archimedes and Euclid did more than all the bookkeepers of Great Britain and the United States put together, the latter should abandon Arabic numerals, the concept of zero, and the principle of positional notation to go back to the system of Archimedes and start drawing diagrams in the sand. Instead of suggesting such a change we may well wonder how much greater the output of the Greeks would have been had they not been handicapped by their cumbersome methods.

In his comments on the variability of the vara and the awkwardness of the hundred-meter tape in laying out railroad curves, Mr. Jordan is, in the main, correct. But he leaves himself wide open for a host of *tu quoque* arguments. How about gallons, American and Imperial? Bushels, American and British? Ounces, fluid, apothecaries', and troy? The miner's inch of California and of Southern California, Idaho, and Colorado, not to mention other varieties? Pounds, troy and avoirdupois? Tons, short and long? Hundred-weights of 112 lb and stones of 14? But let it go at that.

Mr. Jordan exaggerates the danger of making errors in locating the decimal point in the metric system. Would he, for fear of making similar errors, advocate abandoning the American monetary units of dollars and cents to go back to pounds, shillings, and pence which are the blood brothers of feet and inches? Would he prefer to say that the average weight of a corn-fed crew from the Middle West is 13 stone 3 lb in the good old English way instead of 185 lb? Is Mr. Jordan afraid of misplacing the decimal point when he uses 9 as the characteristic of a logarithm to avoid using -1? If he is not, he need not be afraid of getting meters and millimeters mixed up in reinforced concrete design.

In closing, the writer may be permitted to remark that it is a sad commentary on human nature that, when discussing objective scientific subjects, even engineers cannot get away from narrow prejudiced nationalistic viewpoints. The discussions are most likely carried out with a heated fervor worthy of the theologians of the Middle Ages.

ADOLPHO SANTOS, JR., Assoc. M. Am. Soc. C.E.
São Paulo, Brazil

The Dilemma of Engineering Education

TO THE EDITOR: It is to be expected that the current survey of civil engineering education being undertaken by the Society's Committee on Engineering Education in cooperation with the Civil Engineering Division of the Society for the Promotion of Engineering Education will be an important step towards the improvement of curricula. A questionnaire such as that reported by Roy M. Green, in the February issue, under the title, "Need for Broad Training Indicated by University of Nebraska Questionnaire," is certainly one of the first steps in the fact-finding phase of the study; but it would be unfortunate if the work of the survey should stop with a tabulation of the results from a questionnaire. Conclusive and comprehensive recommendations will certainly be welcomed by all concerned with the training of engineers.

From answers to the Nebraska questionnaire, it would appear that there are many engineers who support a broader, more "cultural" education for the profession. Quoting from Mr. Green's article, "A well-known authority on flood control uses the following words: 'Like any other professional training, engineering should be based upon a broad non-technical foundation, as thorough as can be obtained without neglecting professional work.'"

The foregoing suggests the dilemma which the present survey of the Society and the S.P.E.E. must solve if it is to prove a real contribution to engineering education. How is the engineering college to provide this "broad non-technical foundation without neglecting professional work?" Where to find room in an already crowded curriculum for courses in economics, history, psychology, English, public speaking, to say nothing of literature, art, music, and foreign languages? What to leave out of the technical training which is already often inadequate?

The dilemma cannot be resolved, I am sure, within the existing framework of engineering education. To return to our quotation, "Like any other professional training, engineering should be based upon a broad non-technical foundation,..." What other professional training includes both its foundation and its professional work in a four-year college course? The training required for entrance into the medical and legal professions is well known, though few engineers would readily admit that their profession is less demanding in technical knowledge than medicine or law.

It may be claimed that the engineering college graduate is prepared, and should be prepared, only to enter a sort of apprenticeship, during which he continues to prepare himself for his profession. But this is also true of medicine and law and, to an even greater degree, in the year or more spent as intern or law clerk.

The only complete solution to our dilemma lies in placing engineering training where it belongs, in graduate schools. The college could then provide the "broad, non-technical foundation" along with prerequisite mathematics and natural sciences. Some steps have already been taken in this direction, the only one with which I am familiar being at Harvard University, where the undergraduate engineering school has been abolished and advanced professional training is provided only in the Graduate School of Engineering.

I should like to see some university go the whole way and set up a graduate engineering school with possibly a three-year curriculum, requiring a bachelor's degree for entrance and leading to the degree of C.E., M.E., or E.E. These professional degrees might then have the same standing that M.D. and LL.B. have now.

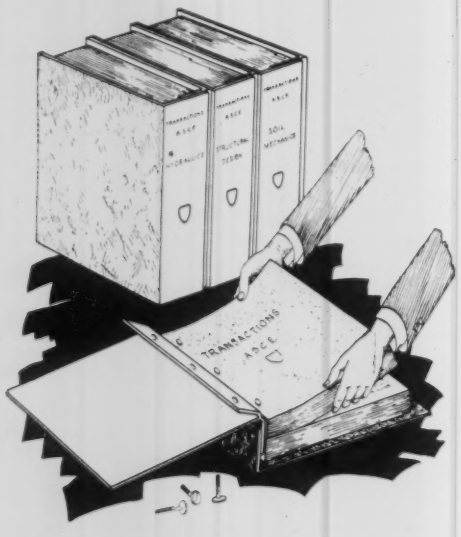
WILSON V. BINGER, Jun. Am. Soc. C.E.
2d Lt., Corps of Engineers, A.U.S.

Greenwich, N.Y.

Distribution of Transactions

DEAR SIR: This letter presents a suggestion which would, it is believed, result in increased usefulness of the valuable technical information contained in TRANSACTIONS. The idea was presented to the Committee on Publications about a year ago, but due to war conditions they decided not to consider the proposal seriously. However, with the end of the war, and the almost inevitable dispersion of our membership to foreign countries, I believe the idea should be brought before the Society for discussion and action.

The suggestion is to send the complete TRANSACTIONS only to those members who request it, but to have reprints of each paper made available for free distribution in a form somewhat similar to that now furnished authors and discussers. Members would be furnished with suitable binders so that their reprints could be kept in a permanently classified file. In this way each member could build up an invaluable collection of papers upon subjects in which he was interested. It would, likewise, be a file which he could, and would, transport from one job to another, without undue difficulty.



POSSIBLE FORM FOR FILING "TRANSACTIONS"

It has been my observation, and experience, that relatively few members fully utilize their TRANSACTIONS. Those who might need the complete set of papers usually have access to a library where a complete file is available, or work in an office which has the entire set of TRANSACTIONS. Most of us have quite a problem providing space for them either in our office or at home, and it is probable that many copies are eventually discarded. This occurs even though the individual may be greatly interested in several papers in each copy of TRANSACTIONS.

The technique of distribution should present no problem to our efficient staff in the postwar period. Whether members would receive papers of the Divisions to which they belonged, or whether they would sign a check list each year, or whether some other basis would be better is a matter that would have to be worked out. The form of the binder would, likewise, require investigation. The accompanying sketch is solely to illustrate the general idea and should not be taken as a concrete proposal. The shape of the binder, for instance, probably should resemble, as nearly as possible, the present TRANSACTIONS.

Even though the relative cost of the proposed plan as compared to that now in effect may be the same, a small difference one way or the other should not be considered seriously. It should be emphasized again that the suggestion is made in order to make the technical information available in a more useful and practical form.

DON H. MATTERN, Assoc. M. Am. Soc. C.E.
Knoxville, Tenn.

Editor's Comment: Periodically, revisions for the handling of Society publications of the general type so well outlined by Mr. Mattern have been posed. There are many valid reasons why they have never been utilized—for example, they would give only a partial or limited technical library to members; the expense would be out of all proportion to the value; the advantage of over-all indexing would be lost; and other methods already available can achieve the same result.

Every civil engineer should have a technical library. TRANSACTIONS comprises the most up-to-date that is available, its value increasing with the years. Periodic indexes add to its usefulness. A complete volume covers not only the engineer's special interests but also the finest of references on other collateral professional subjects.

TRANSACTIONS is the cheapest printing that the Society is privileged to undertake. Practically all the text and illustrative material is already "standing" in type. The volume is fairly large, and the edition also is generous. All this results in a low unit cost. But this is not all—about half the Society members order special bindings, in cloth or leather. While the cost of this work varies from year to year, nevertheless the binding always yields some profit to the Society which is immediately translated into additional service, or lower net cost to members.

As against this, to handle TRANSACTIONS papers largely by means of reprints would add to the unit cost of the complete volume and would particularly add to the office cost of handling individual orders, even in terms of thousands of members ordering. The present mass production and distribution is unbelievably reasonable in cost; individual distribution would be problematic in extent, but inevitably wasteful and expensive.

Reprints of TRANSACTIONS papers with their discussions are always available for separate sale. For the intensive use such as Mr. Mattern anticipates, such reprints are available. Taking the 1944 volume as an example, suppose an interested member wanted 20% of all these separate reprints for his condensed file. The total cost to him would be only about $1.50, or considerably less than the price of the leather binding. This would not require any change in the present method, or any failure to receive the complete volume. For purposes of a complete file and use of the comprehensive indexes, he still would have all the advantages, and at the same time he could set up the type of library Mr. Mattern suggests at a cost more reasonable than the special binding.

These and other similar considerations have always appealed to the Society's Committee on Publications as indicating the advantages of making the complete TRANSACTIONS available to every member. This procedure gives much more in value to the member, and at a cost much less, than by an other method now available.

EDITORS

Education and Psychology

TO THE EDITOR: For several years I have had occasion to observe the eternal conflict that exists between the theoretical and the practical man, between the architect and the contractor, between the inspector and the foreman, between the office engineer and the field man, and between the scientist and the general public. To accomplish anything always requires the mutual cooperation of two or more groups often having fundamentally different training and perspective.

To illustrate—an old railroad employee with many years' practical experience had to replace a washed-out bridge. He was given the assistance of a young engineer. Later, when asked to furnish a report on the progress of the job, he replied, "I don't know if that young feller has the picture drawed yet, but the bridge is up and the trains is runnin' over it." Conversely, many failures can be attributed to the fact that practical men have relied too heavily upon previous experience which was not applicable to the case at hand.

The number of cases in which psychology plays a part are, of course, infinite. A few examples will be cited:

1. The U.S. Coast and Geodetic Survey long ago discovered that monuments and bench marks which were given names of some local significance were much less likely to be destroyed by children or vandals than those given an impersonal number.

2. Many papers, articles, and books are being written, which purport to present to Americans, and particularly to discharged soldiers, so-called postwar opportunities and developments. Many of these, if not completely false, are downright misleading. Recently I read one of these on rammed earth homes. One statement was to the effect that 80% of the soil found in the United States is suitable for rammed earth construction. Where does this figure come from? Another claimed that a certain rammed earth structure was found to be 60 F warmer inside than out. Though the writer did not say so, the uninitiated could certainly draw the conclusion that the rammed earth walls actually give off heat. Further, the whole article gave the impression that any G. I. Joe with a couple of thousand dollars could build one with the help of his neighbors. I would hate to see G. I. Joe lose the few dollars he has saved during the war on such a doubtful proposition. Furthermore, it is quite possible that rammed earth, soil cement, or rammed earth with asphalt binder may prove very practical for small homes. It would be most unfortunate to have a possible new development ruined or distinctly set back because of failures caused by an ill-informed or misinformed few.

3. Because of the enormous number of young Americans who are being exposed to the use of the metric system of measurement, we may expect that efforts will be made to obtain its adoption here after the close of the war. Certainly most scientists and engineers are agreed that the adoption of the metric system would simplify and expedite a great deal of work and the exchange of ideas between nations. However, the ground work must be carefully prepared, and the sponsors must tread lightly or their efforts will come to naught. It is no easy matter to change the habits of decades and centuries, to unlearn facts acquired with some difficulty, and to overcome mass inertia.

The engineer can eliminate a great many difficulties, if he takes proper recognition of the point of view and background of the particular group he happens to be dealing with. The matter was brought sharply to my attention over four years ago by my commanding officer. He said, "You will not give orders so that they can be understood; you will give them so that they cannot be misunderstood."

JOHN E. SHEA, Assoc. M. Am. Soc. C.E.
Captain, Corps of Engineers,
Jacksonville, Fla. *U.S. Army*

Metric System Unnecessary

TO THE EDITOR: In recent issues of CIVIL ENGINEERING several writers have advocated discarding our measuring system in favor of the metric units. Most of their dissatisfaction arises from the way we divide the inch, rather than from the use of the foot. If we could devise a smooth-working method of handling the inch and its divisions, we would be free of the danger of having to make the complete shift to the metric units.

The foot seems to have originated with the Romans, being two-thirds of the old cubit, which was too large for convenient usage as a unit. The Romans further divided the foot into 12 parts—uncials, or inches, as we translated it. These inches were further divided into twelve parts, a trace of which is found in the present printers' measure, in which the inch is divided into 72 "points."

Without directly adopting this 12-point division of the inch, we can borrow enough from the general idea to make our usage much more workable. The commonly used divisions of the inch may be expressed in these three ways:

COMMON FRACTIONS	DECIMAL PARTS	UNCIAL PARTS
1/2	0.500	:600
1/4	0.250	:300
1/8	0.125	:160
1/16	0.0625	:090
1/32	0.03125	:046
1/64	0.015625	.023

In the uncial division the digits have the inch to foot fractional value. Supplying a single symbol for 10 and a single symbol for 11, we are able to write any expression involving 64ths of an inch with only three figures—that is, $-0 - {}^{57}/_{64}$ in. = 0 in. plus $^{3}/_{4}$ in. plus $^{1}/_{8}$ plus $^{1}/_{64}$ = :900 plus :160 plus :023 = 0 ↖:83 in., the single symbol for 10 being a line in the 10 o'clock direction with a barb on the right side, while for 11 the barb is on the left side, as for all odd numbers. These clock-face figures are easy to comprehend and make.

Fractional inches reduced to these uncial terms can be readily added or multiplied and converted to feet by moving the uncial point one place to the left, as in decimals. As an example to find the rise for a third pitch roof at 10 ft 7-$^{9}/_{16}$ in., convert to uncials ($^{9}/_{16}$ in. = $^{1}/_{2}$ in. plus $^{1}/_{16}$ in. = .600 plus :090) = ↖ : 7690 ft
and multiply by the rise per ft, or 8 in. = : :8
7:10600 ft
= 7′ − 1-$^{3}/_{64}$ in.
= 7:1069 exactly

This particular type of problem can be more easily handled by using Smolley's Tables, of course, but the general idea is here advanced with the thought that every engineer has a different background of work experience, and some may find a useful place for it. For instance, those engineers who habitually work with building plans and construction might consider running their levels with an uncially divided rod, thus making it easy to get an exact setting for elevations expressed in sixteenths. In the mechanical field, these uncial numbers might provide a means of size and gage identification that would result in a simplification. An uncially divided micrometer should be considered.

The arithmetic involved seems very slow and awkward to us, but that considerable segment of the world's civilized population that uses British coinage does the same arithmetic quickly and easily as a matter of course. All our old land records and building plans have the foot as a common denominator, and we would avoid a lot of confusion if we could perfect our system so that it would not be vulnerable to attack by exponents of the metric system.

Detroit, Mich. PAUL VAN BUSKIRK, M. Am. Soc. C.E.

Forum on Professional Relations

CONDUCTED COLUMN OF HYPOTHETICAL QUESTIONS WITH ANSWERS BY DR. MEAD

Herewith Dr. Mead gives his answer to Question No. 35, which was announced in the August issue. The question states that: "A university student, enrolled in the electrical engineering course, is offered part-time employment with one of the telephone companies associated with the A-system about a year before the time for him to graduate. The representative of the company tells the student that the company intends to help him through school by giving him employment during the following school year on a telegraph repeater maintenance job, and that after graduation it plans to put him in the engineering office as a repeater man. The work given him while in school is light and involves easy hours, and he gets well paid for doing it as it is of a special nature. He is benefited, and the A-company also receives full value for the service he has given while holding the job. After the student graduates and goes through the company's special six-week course in order to learn more about the repeater work, he is offered a good position with rather unusual opportunities in another company. Should he accept, or what would he do in this case?

To the writer the conditions seem to indicate value received on the part of both the student and his employers. The fact of his employment seems to indicate that he is expected to continue his services, and there may be—and perhaps should be—a sense of obligation to do so. However, the writer does not see that the student is ethically bound to stay with his employers provided his opportunities in another direction are superior.

DANIEL W. MEAD, Past-President
and Hon. M. Am. Soc. C.E.

Madison, Wis.

Question No. 36, which was announced in the September issue, will be answered in the next, or November, number. Replies to the following question may be received until November 5, with answers in the December issue.

QUESTION NO. 37: *A few years ago a student was working on an engineering crew, which had charge of a grade-separation project. Occasionally the contractor would send a box of cigars and a carton of cigarettes over to the office. These were accepted. Did the crew violate any ethical consideration?*

SOCIETY AFFAIRS

Official and Semi-Official

Secretary Carey Attends Meeting of Los Angeles Section

ABOUT 350 members and friends of the Los Angeles Section attended the first dinner meeting of the season on September 12 at the University Club. One of the subjects for discussion at this dinner meeting was Society activity in engineer welfare matters, and particularly with regard to collective bargaining. On hand to lend counsel was Secretary William N. Carey, who had made the trip to Los Angeles at the special request of the Section. In an informal address at the meeting, he stressed the predominance of Board interest in the human side of engineering and the economic welfare of civil engineers. He also outlined the history of the Society's activities in relation to collective bargaining.

Two days later, on the return trip, the Secretary stopped at Salt Lake City, Utah, to attend a regular scheduled meeting of the Intermountain Section. The scheduled discussion was on the sanitary engineering problems of the state. Here the Secretary outlined again the same subject he discussed before the Los Angeles Section.

A second stop on the return trip was made at Chicago, where Secretary Carey attended a luncheon meeting of the Chicago Engineer's Club "Hot Stove League." The league assembles frequently for exchange of news, gossip, and good fellowship. Several members of the Society are active in this most informal group.

Secretary Carey reports that there appears to be no relief as yet in travel conditions. Trains and hotels are filled to capacity consistently, and fortunate indeed is the man who obtains the accommodations for which he has made reservation in advance. Notwithstanding travel difficulties, the Secretary has been encouraged by the response at the several meetings he has attended since his appointment, to plan visits to each and every Local Section just as soon as practicable. He hopes to learn more of the local interests, objectives, and needs of Society members throughout the nation and thus improve the ability of our Headquarters to give better service to all.

NOTES FROM THE CAPITAL

Occasional Information Transmitted by the Society's Washington Representative and Believed to Be of Special Interest to Civil Engineers.

Guide to Industrialization of China Prepared

THE Engineering Service of the Foreign Economic Administration has prepared a ten-volume *Guide to the Industrialization of China*. Intended to serve as a technical aid to the Chinese in developing their industrial plan, the guide was made available to the Chinese Government and American Government officials in February 1945. A condensation of the *Guide* is now being prepared for general American use.

Outlined in the plan are nearly 600 industrial establishments and procedures for improving and expanding transportation systems at a cost of approximately $1,900,000,000. The study is neither an official program nor an attempt to indicate the degree of industrialization required to meet all of China's needs. Since the *Guide* is essentially an engineering study, it does not deal with the mechanics of financing the program. The major objective in preparing the *Guide* was to indicate the kind and the extent of the basic industries which must be developed in order to establish the essential elements of an industrial structure. Perhaps more important even than the industrial and technical data which it presents is the fact that it defines a starting point for industrialization and outlines a pattern of integrated industries which should be developed before a more ambitious program can be undertaken.

In their work, the engineers of the Foreign Economic Administration were assisted by leading American industrial and engineering firms in determining the manufacturing processes and physical characteristics of plants which could be adapted to Chinese conditions. Industrial training and hygiene programs were included to assist the Chinese in meeting the manpower requirements of industrial expansion. Leading American universities and the U.S. Public Health Service assisted in this part of the work. In the development of the *Guide*, the FEA engineers had the cooperation of the Washington representatives of both the National Resources Commission of China and the Chinese Ministry of Communications, as well as of many Chinese engineers in this country.

An abstract of the *Guide* prepared by the FEA states:

"The *Guide* was undertaken in the belief that the emergence of China as a modern industrial nation, and the technical and economic assistance of the United States in achieving that end, will prove to be of substantial benefit to both countries. The Chinese lack so many of the basic necessities of modern living that their potential requirements for clothing, housing, medication, and the manifold products of an industrial age challenge the imagination.

"The 450 million people of China, representing one-fifth of the world's population and occupying an area over a third larger than the area of this country, need the tools of production and the technical know-how for coping with twentieth-century problems. The most important contribution that can be made by the people of the United States to accelerate this development is to enlist our technical ability and knowledge in assisting the Chinese to plan and build their industrial plant. . . .

"The end of hostilities in Europe, and the Allied victory in the Pacific have focused the attention of the entire world on the problems of the peace. There is a growing conviction that the industrialization of the great undeveloped areas of the world, and especially of China, should be encouraged and aided. Technical guidance in the planning stage will not only keep the doors open for American industry, but will create good will in our future political and economic relations with other countries and will contribute to the establishment of a sound basis for economic recovery and world peace.

"Although the *Guide* offers only one of many possible plans for initiating China's industrial development, it was based on the conviction that the most practical and effective approach to the problem of establishing new industries in China is to provide a wide enough range of product and technology to meet the needs of an expanding industrial economy and to develop the know-how for further expansion. The *Guide* was prepared with this objective and will serve its purpose if it is used for the development of a well-rounded, balanced, and operative industrial plan."

Also the abstract states that:

"It was recognized that a broad plan of development for China should encompass programs for:

"1. The establishment of industrial centers throughout China proper
"2. Rehabilitation of war damage
"3. Conversion of Manchuria's war industries to peacetime industries
"4. River and hydroelectric development on the TVA pattern
"5. Expansion of export trade

"The scope of the plan presented in the *Guide* was limited to the first of these five programs because of the conviction that China's prime need in developing an industrial economy is the establishment of basic industries upon which industrialization depends. Although their ultimate importance is recognized, development of most of the other programs will depend upon information which will not be available until the final stages of the war in the Pacific, or which will require more detailed surveys of resources and conditions in China than can be undertaken at this time. . . .

"On the other hand, the TVA type of combined river control and hydroelectric and industrial development, which will be of considerable importance in the future development of China, requires detailed regional study and many years of planning and construction. Preliminary studies of this type of development have been made by Dr. John L. Savage, Chief Engineer of the Bureau of Reclamation, who was sent to China by the Department of State to inspect various projects and potential sites. Although the *Guide* does not deal with the problems of large-scale hydroelectric development, it does include a discussion . . . of river control."

West Is Planning for Construction

A TOUR of our Local Sections on the Pacific Coast during the month of August revealed to the director of the Committee on Postwar Construction, Oscar L. King, Assoc. M. Am. Soc. C.E., that this section of the United States is decidedly cognizant of the importance of planning for construction in order to offset the possibility of unnecessary unemployment. The attendance at the meetings scheduled was indicative of the importance of this subject, because those who came took an active part in the meetings. Their suggestions and criticisms have formed a pattern that could well be followed in other sections of the country.

While in nearly every city visited public works construction in the postwar era was high on the list, private construction, if listed at all, was low in the volume already planned. True, this is due to the reluctance of private enterprises to divulge their plans before the eleventh hour, but it may also be because there is a serious lack of such projects. The aims of the Committee on Postwar Construction have always been to bring this volume of private construction up to reach ten billion dollars for the first postwar year.

Although the term "postwar" was correct when Mr. King started on this trip, before it was consummated we had entered the threshold of the period of reconversion. Perhaps now there will be a sharp upward curve to the graph depicting private construction, which is what we sincerely hope will occur.

From his visits to Utah, Idaho, Washington, Oregon, and California, Mr. King learned that all these states are anxious to get going on the construction survey, which will give a fairly true picture of the volume of private construction planned for these states. He also found a general lack of skilled workers and a scarcity of engineers, draftsmen, etc., for preparing plans now.

Idaho is most anxious to confine its construction to private rather than public works, realizing that if private construction takes its proper place, public works of necessity will follow in due time. This is vastly important because if a state is reluctant or lax in planning for private construction, the door will be opened wide for the admission of another made-work program, and all fully realize the costliness of such a program for a minimum of needed and useful structures.

Now that the war with Japan has come to a close, it will be interesting to watch the statistical graph to determine the progress of private construction. Will it be a healthy progress, or one that will be retarded and result in the abuses of made-work programs?

The responsibility for this story rests on the shoulders of each individual as to whether or not he is doing his part to promote this idea locally—that is the place where it must originate because it is the "local" man who knows his local needs best, and his knowledge of this will help him directly as well as his community.

If prosperity comes to his community, he will be benefited. And the reverse is also true—if unemployment and depression visit his community, he and his neighbors will be affected thereby.

The Engineer in Foreign Service

XXV. Refuses Steak, Life Saved

By SGT. ROBERT M. OKEY, SON OF CHARLES W. OKEY, M. AM. SOC. C.E., PRINCIPAL CIVIL ENGINEER, TVA

I LIVE in a villa overlooking the Mediterranean, formerly occupied by a paramour of the King of Spain. Now working very little. All I do is supervise the labor of German prisoners.

I had a nice little airplane ride from Liége to Paris last winter. Liége, by the way, was where I was in a field hospital, and it also happened to be the center of the buzz bomb attacks. You should feel those babies go off. Sometimes they would go over every 5 minutes, and at a mile away they can just about knock you out of your bed. Their destruction was enormous for one bomb. After heavy shells for several days running past over my head this was mild, but after lying there and listening to the motor cut off I was worrying about them too.

I can tell you one more. I probably had my life saved by refusing to eat a steak. Sounds queer for me, doesn't it? Well, I was in a room in a house in a little Belgian town that was under spasmodic artillery fire. My platoon guide (38 years old and an ex-butcher) was just cutting up a cow we had appropriated and asked me if I wanted some. After having had nothing but cold beans for three days I don't know why I refused but I did. I went next door and while I was there an "88" zeroed in on the cross-roads there, and a shell went into the room I had just left and exploded inside, and this fellow caught a piece of shrapnel right in the head. I don't know how he ever came out. I went down in the basement of the next house while they almost shelled it from over us. We just sat there with the mortar trickling down our necks. I finally ran out and got on a jeep, between shells, and went back to my battalion, so I got out okay. Not many of my company did. Three days after I left, our first platoon had a strength of 6 men out of 40.

(*Reprinted from the July 1945 Tennessee Valley Engineer," publication of the Society's Tennessee Valley Section.*)

Sacramento Section Studies Railroad Problem

IN THE CITY of Sacramento, Calif., a serious traffic problem exists because of the presence of railroad tracks bisecting the city. Various interests in the city have indicated their desire to attempt a solution. Opportunity to serve the community, in a field where their engineering services would be particularly valuable, was recognized by the Sacramento Section of the Society.

In 1944 appointment of a committee to study the so-called "Western Pacific Problem" was approved. Frederick W. Panhorst was named chairman, and the following were included as members: Conrad A. Ecklund, Fred J. Grumm, Carl M. Hoskinson, Carl Maughmer, Gilbert F. Mellin, Stewart Mitchell, A. G. Mott, George G. Pollock, N. C. Raab, Henry M. Rich, R. Robinson Rowe, Norwood Silsbee, J. G. Standley, Jr., Edwin F. Sullivan, and Edward E. Welch—all members of the Society.

The committee has been active in the prosecution of its work since its formation, considering the two general solutions to the problem: (1) moving the track to a new location, and (2) improving conditions with the track in its present location. Each general solution embodies many alternatives which are being studied by the committee. Two progress reports have been presented by the committee, and members of the Section have been given opportunity to question the committee and to make suggestions. The committee expects to complete its report within the next few months and to submit it to the Section for such action as seems appropriate.

Appointments of Society Representatives

BORIS BAKHMETEFF, M. Am. Soc. C.E., has been appointed the Society's representative on the Engineering Library Council of the Board of the Engineering Societies Library, the appointment becoming effective October 1, 1945.

E. E. BAUER, M. Am. Soc. C.E., has been appointed the Society's representative on the Committee on Specifications for Sieves for Testing Purposes, Z 23, of the American Standards Association; L. D. DRAPER, M. Am. Soc. C.E., the Society's representative on the Association's Committee on Standards for Drawings and Drafting Room Practice (Committee Z 14); and GEORGE A. THOMPSON, Assoc. M. Am. Soc. C.E., the Society's representative on the Association's Committee on Loading Platforms at Freight Terminals and Warehouses (Committee E-12).

R. H. BURKE, M. Am. Soc. C.E., has been appointed the Society's representative on the Washington Award Commission, his appointment being effective as of May 31, 1945.

ABEL WOLMAN, M. Am. Soc. C.E., has been appointed the Society's representative on the Division of Engineering and Industrial Research of the National Research Council for the coming three-year period beginning July 1, 1946. He will succeed FREDERICK H. FOWLER, Past President Am. Soc. C.E., whose term expires on that date.

News of Local Sections

Scheduled Meetings

CINCINNATI SECTION—Regular meeting at the Cincinnati Engineers' Club on October 2, at 8 p.m.

CLEVELAND SECTION—Dinner meeting at the Cleveland Engineering Society on October 19, at 6:30 p.m.

DAYTON SECTION—Luncheon meeting at the Engineers' Club on October 15, at 12:15 p.m.

GEORGIA SECTION—Luncheon meeting at the Davison-Paxon Tea Room on October 5, at 12:45 p.m.

LOS ANGELES SECTION—Dinner meeting at the University Club on October 10, at 6:45 p.m.

MARYLAND SECTION—Dinner meeting at the Engineers' Club on October 18, at 7 p.m. (Cocktails at 6 p.m.)

METROPOLITAN SECTION—Technical meeting in the Engineering Societies Building on October 17, at 8 p.m.

MIAMI SECTION—Dinner meeting at the Southern Tavern on October 4, at 7 p.m.

NEW MEXICO SECTION—Technical meeting at the University of New Mexico on October 17, at 7:30 p.m.

NORTHWESTERN SECTION—Dinner meeting at the Minnesota Union on October 1, at 6:30 p.m.

SACRAMENTO SECTION—Regular luncheon meetings at the Elks Club every Tuesday at 12 m.

SAN FRANCISCO SECTION—Dinner meeting at the Engineers' Club on October 16, at 6 p.m. (C. G. Gillespie will be the speaker on a topic related to postwar sanitary projects in the San Francisco Bay area.)

TENNESSEE VALLEY SECTION—Dinner meeting of the Knoxville Sub-Section at the S & W Cafeteria on October 10, at 5:45 p.m.

TEXAS SECTION—Luncheon meeting of the Dallas Branch at the Adolphus Hotel on November 5, at 12:15 p.m.

TRI-CITY SECTION—Dinner meeting at the Muscatine Hotel, Muscatine, Iowa, on October 11, at 6:30 p.m.

Recent Activities

LOS ANGELES SECTION

The annual high jinks of the Los Angeles Section took place at the Oakmont Country Club on July 25. The party consisted of a golf tournament, which started at noon, and a dinner and entertainment in the evening. The golf tournament, managed by Roy Anderson, attracted the largest number of contestants on record, with 23 teeing off. The prizes, which were donated by Don Warren, went to Lt. Ray Sparling, U.S.N., low gross; David Fiscus, low net; and Mason Lance, biggest jinx. A large gallery followed the contestants over the course, giving them encouragement or jeers, as the case might be. Musical entertainment was provided during the cocktail hour and dinner, a featured number being a duet by President Salsbury and an accordionist. There were, also, dancing acts, songs, and a rousing xylophone solo by a blind artist. There was the unusually large attendance of 165, and John Albers and Roy Anderson received a vote of thanks for their efforts in making the day a success.

NEBRASKA SECTION

On July 24 the Section held a special dinner meeting in Omaha in honor of Society President John C. Stevens. The attendance of 54 included several of Mr. Stevens' classmates from his school days at the University of Nebraska. In his talk Mr. Stevens discussed some of the Society's activities, and the purpose of the Board in sponsoring the work described. He stressed, particularly, the work of the Committee on Engineering Education, the Committee on Postwar Planning, and the Joint Committee on the Economic Status of the Engineer. Mr. Stevens also touched on the rôle of the Society in the preparation of a plan for the industrial control of Germany and Japan.

PANAMA SECTION

The Panama Section held two meetings in August. On the 6th there was a talk by F. E. Hulse, chief engineer for F. H. McGraw and Company of Panama. In his talk, which was entitled "Trends in Airport Design," Mr. Hulse gave a résumé of airport design from 1920 to date, emphasizing especially military influences and touching on postwar possibilities. On the 13th, there was a joint meeting of the Section and the Canal Zone post of the Society of American Military Engineers. The feature of the occasion was the showing of the Allis-Chalmers Manufacturing Company's sound film, "Tornado in a Box," which concerns the continuous combustion gas turbine. There is considerable interest in technical circles in the recent development of the gas turbine, and the group was enthusiastic at the opportunity to see the film.

PUERTO RICO SECTION

The Section held its fourth technical meeting of the year on August 22. Principal speaker and guest of honor was Fred N. Severud, New York City consultant, who has been employed to make a survey of the construction industry in Puerto Rico, with a view to suggesting methods for reducing the cost of, and improving the design of, concrete. In this talk, which was entitled "Facts and Fancies in Engineering and Construction," Mr. Severud discussed modern trends in reinforced concrete design, with emphasis on the structural form. During the business session Raoul N. Arroyo was elected secretary-treasurer, replacing Luis M. Guillermety who resigned.

SACRAMENTO SECTION

At the annual Junior Day meeting on August 7 E. G. Burckhardt, a Junior in the Section and assistant engineer for the U.S. Bureau of Reclamation, gave an account of the first railroad in California. The meeting held on August 21 featured talks on Folsom Dam by Frank Kochis and Lt. Col. W. D. Morrison. It is proposed that this dam will form a multiple-purpose storage reservoir on American River, 25 miles northeast of Sacramento. Political topics, presented at the other two meetings, included talks by Gene Desimone on "Propaganda and Government," and Prof. Michael J. Brickley on the San Francisco Conference.

SAN FRANCISCO SECTION

A lecture on the "Construction of U.S. Naval Drydocks at Hunter Point" comprised the technical program at a dinner meeting of the Section held on August 21. This was given by George F. Nicholson, captain, Civil Engineer Corps, U.S. Naval Reserve, who is public works officer and officer in charge of construction at the Hunter Point Navy Yard. The attendance of 175 included several students, who were guests of the Section.

SOUTH CAROLINA SECTION

The annual summer meeting of the South Carolina Section—held at the Columbia Hotel in Columbia on July 26—took the form of a joint session with the South Carolina Society of Engineers. The program consisted of a technical session, business meeting, and dinner. The list of scheduled speakers included R. M. Cooper, director of the South Carolina Research, Planning, and Development Board; Martin S. Flood, lieutenant, Civil Engineer Corps, U.S. Naval Reserve, stationed at the Congaree Air Base; and Brig. Gen. R. F. Fowler, who is division engineer at Atlanta, Ga.

TEXAS SECTION

There was a good attendance at the August luncheon meeting of the Fort Worth Branch. Following luncheon and a brief business session, there was a general discussion on the city-wide clean-up campaign. The principal speaker—Henry Cook, until lately a lieutenant in the U.S. Naval Reserve—was then introduced. Mr. Cook described the work of the Seabees in general, and drew attention to the part played by pontoons in winning the war.

TOLEDO SECTION

On July 24 members of the Toledo Section of the Society participated in a meeting of the Toledo Technical Council, of which the Section is one of the sponsoring groups. The technical program consisted of a talk by William C. Eddy, captain, U.S. Navy, who is officer in charge of Navy radar and radio training. Touching briefly on the important uses of radar, Captain Eddy pointed out that there are many special uses of this valuable equipment, and that some of them are still closely guarded secrets. He went more into detail on the subject of television.

ITEMS OF INTEREST

About Engineers and Engineering

Flood Control and Diversion Works in Persia

By L. M. WINSOR, M. Am. Soc. C.E.
DIRECTOR GENERAL, IRRIGATION ADMINISTRATION, MINISTRY OF AGRICULTURE, IRAN

IT SEEMS that very few people in Teheran know about the River Kahn and the very important place it fills in the agricultural life of the community. It is the source of water supply for all the lands lying on the west and south of Teheran between the villages of De-Rasht and Rey. No one seems to know when water was first diverted from this stream to the lands in this vicinity. But certainly the ancient city of Rey, or Shahabdolazim, utilized this source of water supply to its full capacity; hence it is safe to say that this diversion system has been in constant operation for more than fifteen hundred years.

It is evident that the ancient river bed ran very close to the village of De-Rasht, thence southeast along the southwestern border of Teheran. It is my conclusion that a large canal was opened up many centuries ago, running due south from a point south of the village of Kahn, where the present head of the Feruzabad Canal is located, and where the Karaj-Teheran kanat now crosses some 30 m below the stream-bed, and that heavy floods broke over the river banks and washed out the diversion canal to make a new channel where the river now flows. Since that time it has been necessary to divert the water back into the canal that runs near the old river bed, in order to convey it to the irrigated lands west and south of Teheran.

During all these centuries, there seems to have been no attempt at building a permanent diversion and control dam. Instead, the water users have followed the age-old practice of carrying boulders into the stream at flood time in order to keep the water flowing into the canal. This required daily maintenance because the stream, which is fed by melting snow, fluctuates very greatly as between night and day, and from day to day. Hence the work of an entire day would often be lost and would have to be repeated in part or entirely on the following day in order to keep the water in the canal. When the heavy spring runoff was over, sand would be shoveled or carried into the rock diversion dam in an attempt to close the leaks. But this practice was never satisfactory and resulted in much loss of water.

Likewise the canal for several kilometers was subject to heavy erosion by floods. Many breaks occurred, and each of these, when repaired with willow baskets filled with boulders, would leak badly, again causing much loss of precious water from the canal.

In 1945, instead of repeating these annual practices, the Irrigation Administration has used the annual maintenance fund in building a permanent control works that will last, not only for ten or twenty years, but for many centuries. In fact it is probable that it will continue in service with very little maintenance even for thousands of years, for it is built entirely of river-bed stone with the strongest kind of concrete filling the voids.

GRAVEL-CONTROL SPILLWAY BEFORE VOIDS WERE FILLED AND POINTED

The natural, water-worn boulders are placed close together, without mortar in the exterior surface but with all voids filled with concrete on the inside of the wing walls, and beneath and between the lower ends of the stone in the floor or apron of each spillway structure. In the foundations of the walls and in the downstream toe of the spillways, large boulders up to half a cubic meter in volume were placed in the bottom of excavations that extend $1^1/_2$ m below the stream bed, or down to huge boulders that were buried beneath the surface.

In the walls above ground, double rows of large boulders, one-man size, were placed with the long axis of each boulder pointing inward and slightly downward, and with the thick ends of irregular boulders on the exterior. Care was exercised to place each boulder in such a way that it fitted into the point between two or more rocks already placed. No small rocks were used at any time. The exterior of the wall has the appearance of having been laid without mortar of any kind, for the concrete was placed only on the interior to fill all voids. Care was also exercised to keep the exterior surface absolutely clean. Hence it presents a very attractive appearance as well as one of strength and stability.

The floor or apron of each spillway was laid by placing the boulders shingle fashion with their long axes slanting downward and with their thick ends on the upper surface. Concrete fills all voids but does not separate the stones one from another. This makes it virtually impossible for the structures to fail. None but hard, solid, river-worn boulders were used in any part of the structures. Repeated floods of high water have tested the work most thoroughly and no weakness has been found.

It is interesting to note that the workmen were without exception just common,

SPILLWAY FOR FLOOD CONTROL AND GRAVEL CONTROL ABOVE KAHN RIVER DIVERSION

SECTION OF WING WALL AT DIVERSION DAM IS 5 M HIGH, 200 M LONG, 1 M THICK AT BASE AND 60 CM THICK AT TOP

unskilled laborers, without previous training in work of this kind. One laborer appeared who had worked previously as a stone mason, but at the end of the first day he was discharged because he insisted upon using his own ideas, which were contrary in every respect to the system prescribed. He wanted to lay the stone in courses as in a brick wall, with the long axis of each stone parallel to the exterior face of the wall, and with small stones to level each rock, also with mortar between. He had too much to forget; hence his services could not be used.

The writer gave his personal attention to the work of training the men to do a good job. He taught them how to obtain the best possible mix of concrete, using approximately one part of cement to three parts of sand and six parts of coarse aggregates (which included a splendid gradation of gravel up to 10 cm in diameter), with clean water sufficient to make a plastic mix. Each batch was thoroughly mixed. It was necessary at first to remove and replace much of the work done by the men while they were learning the technique of laying rock; but with patience and kindness they were made to understand why the work they had done was not exactly right. If they used an excess of concrete the writer required them to remove it and showed them repeatedly how the work should be done. If they spilled concrete on the outside surfaces he required them to scrub the rocks clean, with water and their hands, until they learned by experience that they must keep the exteriors clean.

It required but a few days to discover the men who could do the best work. The others were used as helpers. After five days it was seldom necessary to make corrections. The personal attention and the patience exercised during those first few days has paid big dividends in obtaining efficiency, and in securing a job that would be a credit to skilled workers of the highest grade. Furthermore, after the first two weeks the men could be left without supervision for hours at a time, and the work went forward without the slightest difficulty or delay. We now have a crew of thirty to fifty men that can be trusted to carry on work of much greater magnitude when the time comes for executing the more important jobs that are under contemplation by the Irrigation Administration.

The essential features of the Kahn River project were completed between February 1 and March 15, 1945, at a cost well within the annual budget for maintenance, which amounts to approximately 150,000 rials. This included the main diversion dam and spillway and the diversion canal intake. However, additional works and refinements were planned and executed that are just now being completed at a cost of 150,000 rials additional. These additions include a protection works consisting of dike revetments and a spillway to protect the municipal kanat lines from damage by floods. These works also provide a means of controlling the gravel and sand that normally flowed into the canal system during periods of high water.

They also include a provision for developing power for washing, elevating, and screening this sand and gravel, (of which an immense quantity of the highest quality comes down annually with the spring floods), and a large receptacle for storing these aggregates until they are used. They can be made available for commercial use in Teheran, or a works for concrete products can be established at the site to utilize this great volume of material brought down by the water.

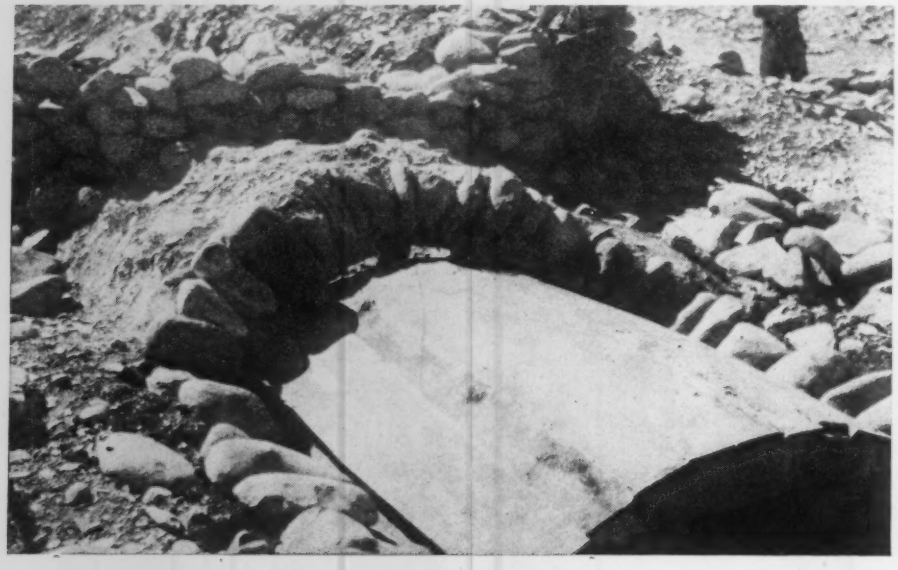
In addition, the delivery canal has been repaired so that there is practically no visible loss of water. It is planned to improve the canal all the way to its destination as soon as funds and equipment are available.

Briefly summarizing, the Kahn River Diversion works has been completed at a cost equal to the cost of annual control for two years, and the maintenance cost in future should not exceed 2% of what was formerly the normal annual expense. Moreover, provision has been made for eliminating sand and gravel from the canal and for storing it where it can be used for commercial purposes and for building. It can supply the annual needs of Teheran for all time.

Furthermore, flood protection has been provided for the municipality's kanat lines. The sections under the river had previously been filled by overflows which caved in and destroyed wells leading from the river-bed surface down into the kanats. The numerous leaks in the main canal have been closed and the banks have been raised and strengthened from the diversion dam to the state highway. Last, but not least, a crew of splendid workmen has been trained and made ready for carrying out the program of irrigation development throughout Iran, plans for which are now ready or in process of being completed.

It should be mentioned that the diversion and control works as completed contain a volume of masonry amounting to 611 cu m; and the excavation and embankments contain a volume of 4,093 cu m. The wing walls are approximately 1 m thick at the base and 60 cm at the top, where the walls are from 3 to 6 m high above the foundation. In some cases the wing walls are only one stone (30 to 50 cm) thick. Cement used averaged only one bag per cubic meter of finished wall. The total cost for the rubble-concrete masonry is only 10% of that for cut stone, which has been used so extensively in Iran.

The embankments were made principally by dragline or power excavation, of which we have one unit, and we have had the use of one small bulldozer (caterpillar-tractor power unit). We have not accepted the war or the lack of equipment as an excuse for idleness. We are endeavoring to use the materials that Providence has provided in abundance, and put them to the use for which they were intended, rather than to push them aside to make room for inferior materials, such as cut-stone masonry or reinforced concrete.



CULVERT BRIDGE 2 M HIGH AND 2 M WIDE, IN PROCESS OF CONSTRUCTION
Long Axis of Stones Points Toward Center of Arch or Quarter Circle

Legal Aspects of Controlled Access to Express Highways

LEGAL ISSUES involved when access to streets and highways is restricted in order to provide better accommodations for through traffic are discussed in a booklet issued by the Public Roads Administration. The report, written by David R. Levin, of the Financial and Administrative Research Division of Public Roads, is entitled "Legal Aspects of Controlling Highway Access."

The author points out that access to certain streets and highways must be controlled to permit the free movement of large numbers of motor vehicles safely and conveniently. Uncontrolled access to express highways, he adds, would defeat the purpose for which they are constructed. He discusses litigation in which property owners have sought compensation because of restricted access, and reviews judicial rulings governing abutting property owners' rights of access, and "right of view."

While respecting the fundamental rights of property owners, courts also should

give careful consideration to the rights and convenience of the traveling public, he suggests. Many courts have ruled that "circuity of travel" is not legally compensable, since it is an inconvenience shared in common with the general public and does not substantially impair access to main thoroughfares. This principle, the author contends, should be applied in litigation that may arise as a result of the construction of urban expressways and controlled-access highways in rural sections.

In view of the huge volume of highway construction planned for postwar years, including many controlled-access thoroughfares, the booklet should be of interest to all state highway officials and to members of the legal profession who handle highway litigation. Copies can be obtained from the Public Roads Administration, Washington 25, D.C., on request.

N. G. Neare's Column

Conducted by

R. ROBINSON ROWE, M. AM. SOC. C.E.

THE OCTOBER meeting of the Engineers Club was like a reunion—so many members back in uniform. They were a handsome happy lot, talking some about coral concrete and 6-day airports, but more about postwar jobs. And cold beer.

"We never dragged our beer from the dock," said one Seabee. "Professor Neare should have known better. We bashed in the keg head with a skull breaker and passed the keg around until it was empty."

"And if we had, can you imagine one Seabee dragging while the other one loafed?" asked another. "They'd have tipped it over and bulldozed it into the canteen. So the problem is impossible!"

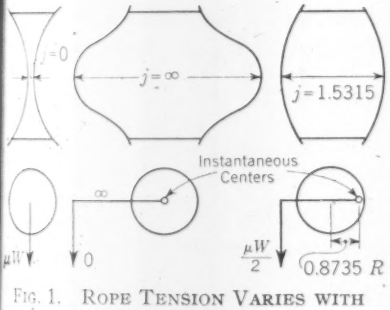


FIG. 1. ROPE TENSION VARIES WITH KEG SHAPE

"Except in my fancy," admitted the Professor. "So I asked if two men pulled directly on tangent ropes whether it could be easier for only one man to pull at a time."

"I'd say not," answered Joe Kerr. "If the weight was distributed evenly over the end of the staves, one man would pull $127 (=400/\pi)$ lb for every 100 he pulled in the team. The cask would rotate with its instantaneous center diametrically opposite the point of tangency of the rope."

"Beer kegs are so rare that Joe has forgotten the bulge," objected Cal Klater. "Suppose the bulge had been negative so the keg was an hour glass; then Brown would have had to pull the same as the team—200 lb. Then suppose the bulge had been infinite; an infinitesimal force would produce enuf torque to move the keg. Somewhere between these limits the bulge is just enuf so that 100 lb will draw the beer. Elliptic integrals stopped me from figuring how much, but the answer is that Brown and Black could be right." (See Fig. 1.)

"That's all I asked and the logic is sound," agreed the Professor, "but I'll write down two equations for computing the bulge:

$$\frac{E - k'K}{1 - k'} = \frac{\pi}{4};\quad j = 1 + \frac{\frac{8}{\pi}E - 2}{1 + k'}$$

where E and K are the complete integrals to mod k, complementary to k', and j is the ratio of bulge diameter to rim diameter."

"Don't those equations neglect the overturning moment of the rope tension, which would vary the base pressure and friction?" continued Cal.

"Right, altho the effect is small. I have computed $k' = 0.06751$, $E = 1.00818$, and $j = 1.5315$ and have seen beer kegs that bulgy."

"For a new problem," continued the Professor, "let me introduce Professor Steinmance, who will exasperate you with the logistics of bringing the Army home."

"I promise to tease, Noah, but not to exasperate. You all know the Army's predilection for order in all things, so you won't be surprised that one general ordered his command divided into equal boat-size debarkation units. The staff proposed 701 men per unit, leaving 11 men to return as casuals. The General disapproved. Then 698 men per unit with only 7 casuals was proposed. The General ordered smaller units, no casuals. His Chief-of-Staff tackled the problem and recommended 612-man units leaving only one casual. 'If we do, you're the man that is left,' threatened the General. But they all came home because a young engineer officer computed the right strength for each unit. Can you?"

[*Cal Klaters were Richard Jenney and Anne Othernut (J. Chas. Rathbun, who modeled the keg with a sad-iron in a sugar jar). Guest Professor Steinmance is D. B. Steinman.*]

New in Education~

"Cultural" Studies Added to Ohio State Curricula

FIVE-YEAR CURRICULA, replacing the present four-year programs in all degree-granting departments of Ohio State University's College of Engineering, will become effective with the autumn quarter, Dean Charles E. MacQuigg has announced. The plan, which will bring more of the so-called "cultural" courses into the engineering curricula, will become effective with the new freshman class. Present students, returning students whose work was interrupted by the war or for other causes, and students transferring here from other colleges, will be permitted to choose between the four- and five-year curricula until two years after the war.

To meet the problem of adequate recognition of the additional competence attained by the student completing the five-year curriculum, Ohio State has provided for a differentiation on the basis of academic standards. Men having superior academic averages at the end of the third year may enter a special program which, if pursued successfully, will lead to a master's degree as well as the bachelor's at the end of five years.

The new Ohio State program will call for 84 quarter hours of fundamental courses; 51 hours of "broadening" studies; 19 hours of required general work, such as military science, physical education, and survey courses; 120 to 126 hours of departmental work—a total of 274 to 280 hours. The 51 hours of "broadening" studies may come from such fields as economics, political science, sociology, economic geography, psychology, business organization, international studies, history, and others.

Lack of earlier action on the part of engineering colleges to include more "cultural" studies was because of inability to reduce the science and technology requirements in the four-year curricula to permit inclusion of the non-technical subjects, the Dean stated. He continued:

"These problems, as well as advice from industry and alumni, have been carefully weighed. . . . The plan adopted at the same time is patterned to meet the needs outlined by the Society for the Promotion of Engineering Education, in a report by the Society's Committee on Engineering Education after the War."

* * * *

Rensselaer Initiates Cooperative Program with Industry

A COOPERATIVE plan with industry has been announced by Rensselaer Polytechnic Institute, to aid in replenishing the nation's supply of scientific and technological personnel, said to be so critically diminished by the war as to threaten the country's future. The General Electric Company has agreed to participate in training as many as 80 men at a time as its share in the program. Other nationally known companies are also arranging to take part.

The R.P.I. plan is described as aiming particularly to fill the need for men holding master's and doctor's degrees. It is said to differ from the average college-industry plan in that it will carry students through at least to the master's degree and enroll only those who indicate talent and ability for graduate study and research. R.P.I. and the cooperating industry will jointly select the candidates from the regular student body in their sophomore year.

The program will get under way November 1. While studying various branches of engineering, physics, and chemistry, students will spend about 30% of their time in shops or laboratories of the cooperating industries, for which they will receive wages.

Large Private Construction Projects in Houston, Tex.

By JAMES W. BRADNER, JR., M. AM. SOC. C.E.
DIVISION ENGINEER, FEDERAL WORKS AGENCY, FORT WORTH, TEX.

THROUGH the exceptional philanthropy of one man, Monroe D. Anderson, and the progressive enterprise of another, Glen McCarthy, the city of Houston, Tex., is assured of two major private postwar construction projects that will cost over 65 million dollars. One of these projects is the Texas Medical Center, estimated to cost over 50 millions; the other is a 16-million-dollar apartment and community center development.

The Medical Center will be administered by a non-profit corporation having a board of trustees. These trustees will be representatives of the foundations and organizations participating in the construction and operation of the various parts of the Center. They will, in general, regulate the architectural design and spacing of buildings, do all landscaping, operate necessary housing and utilities, and coordinate all the efforts being made by two or more of the participating agencies. Almost all the financing for the various parts of the Center has been assured, although at this time the source of some of the funds has not been made public. The final development will be a tribute to the cooperative efforts of many groups, agencies, and foundations, and to the generosity of Mr. Anderson.

It is believed that all buildings will be of reinforced concrete with brick exteriors, trimmed with Texas stone and granite. The architecture might be described as "Mediterranean" to harmonize with the architecture of the existing Hermann Memorial Hospital.

One of the first buildings to be put up will be the 14-story Professional Building, which will be located on part of the twenty acres now used by the Hermann Memorial Hospital. This building will be financed and administered by the Hermann Estate and will cost approximately $1,600,000. Another early development will be additions to the present Hermann Memorial Hospital to increase its capacity from 290 to 1,000 beds. The out-patient department of the entire Center will form one wing of this hospital addition and will be administered by Hermann Hospital. These hospital additions are being financed by a million-dollar gift from H. R. Cullen, by a substantial contribution from the Hermann Estate, and by other gifts.

Another building to be started in the near future is the Baylor University College of Medicine. This college has been given one million dollars in cash by the Anderson Foundation and will be given one hundred thousand dollars each year for the next ten years by the same foundation. In addition, the City of Houston has pledged itself to give the college a hundred thousand dollars each year for the next five years.

The Childrens' Hospital, which will be operated by Hermann Hospital, will contain about 250 beds. There is no lack of money for this particular part of the Center, although the exact details of financing have not been made public. The University of Texas School of Dentistry, which will be one of the most modern in the country, when completed will cost approximately one million dollars. The Anderson Foundation is providing half this amount, and the State of Texas is expected to provide the rest.

The School of Geographic Medicine and Public Health, which will be under the direction of the University of Texas, is to be used for the training of public health personnel. There has been a need for this sort of institution for many years and this need is becoming increasingly evident as the Southwest becomes industrialized and as populations become more and more concentrated in cities and towns. The entire cost of this building is to be borne by the State of Texas.

The M. D. Anderson Hospital for Cancer Research will be built to care for and treat cancer patients as well as to engage in cancer research. It will contain between 300 and 350 beds. The Anderson Foundation proposes to finance half the cost of this building, while the State of Texas will finance the rest. In this connection the State of Texas has been divided into fifteen districts, in which cancer committees have been set up. These committees will select the patients who can be cared for in this new hospital and consult with other patients who cannot secure admission to direct them to proper medical care. This hospital will be operated by the University of Texas with a full-time director thoroughly trained in both clinical and laboratory research.

The Methodist Hospital, of approximately 350 beds, is to be financed by a gift of one million dollars by H. R. Cullen, while the rest is to be contributed by the Methodist Church. The Episcopal Hospital, of approximately 250 beds, likewise will be financed by a gift of approximately one million dollars from H. R. Cullen and a contribution of a million dollars from the Episcopal Church.

A Tuberculosis Hospital of approximately 1,200 beds is also being projected and is now partially financed. This will care for tuberculosis patients from all over Texas. The City of Houston has voted $650,000 worth of bonds and the surrounding counties are to vote the rest necessary to largely complete the financing. It is possible that the Anderson Foundation may also contribute some funds.

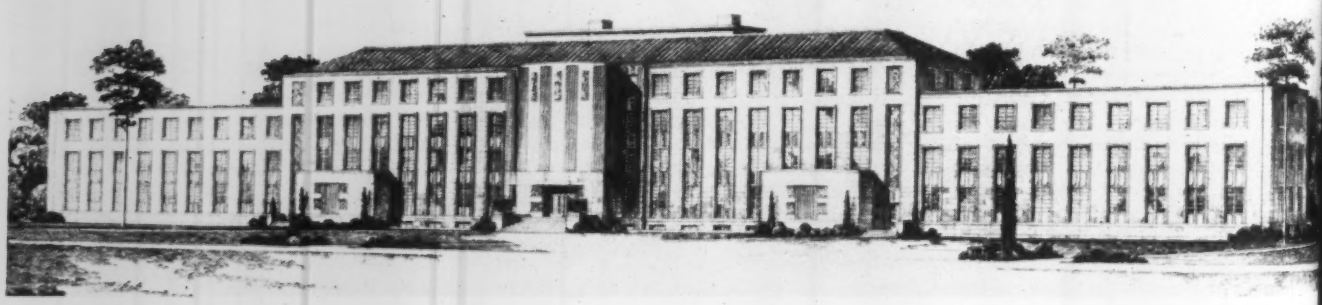
The U.S. Public Health Service is planning a Marine Hospital, which will be operated under its direction. This will be one of its general hospitals operated for the use of all maritime personnel. Another interesting part of the development will be the Convalescent Hospital. It has been completely financed, although the method of financing has not yet been announced. It will contain approximately 200 beds and will be operated by the Center.

Perhaps the largest nursing school in the country will also be a part of the Center. Studies are now being undertaken to determine just what size will be most suitable. This school will probably be operated by the Center itself, with the University of Houston and Rice Institute participating. Financing for this school has been arranged for but not yet announced.

APARTMENTS AND COMMUNITY CENTER

The apartment and community center development is being financed and projected by Glen McCarthy, wealthy Houston oil man. It will be located on a 15-acre site adjacent to the Medical Center. Mr. McCarthy believes that this kind of development will not only fill a long-felt need but will in itself help to create a more modern and progressive Houston.

When completed, the project will include an 18-story building, three 12-story buildings, and two 10-story buildings, all apart-



BAYLOR UNIVERSITY COLLEGE OF MEDICINE, PLANNED AS A UNIT OF THE TEXAS MEDICAL CENTER
Rendering by Hedrick and Lindsley, Houston, Tex.

eight hotels with commercial sections; a group of small buildings with stores, a bank, a movie, as well as underground and overground parking space, a large swimming pool, and a year-round ice skating rink.

The first building to be constructed will be the 18-story building, for which priorities have already been requested. There is good reason to believe that construction of this building will actually get under way in the near future. All buildings are to be of reinforced concrete with brick exteriors, trimmed in Texas stone and granite. They will be completely fireproof throughout. The architecture may be referred to as "Modern."

Members Chosen to Broadcast to Orient

THE dramatic story of the men in the construction field, who have turned the West from a pioneer wilderness into a great industrial arsenal, was told to the people of Australia, New Zealand, and the Orient, by two members of the Society. Recently Stephen D. Bechtel and Harry J. Hilp, San Francisco contractors, were participants in a program recorded by the Office of War Information for Trans-Pacific broadcast.

"The early men of the construction business were willing to try anything. They particularly enjoyed doing the 'impossible,'" Mr. Bechtel declared, citing such water projects as Boulder Dam, which was completed three years ahead of schedule, and the piers for the Golden Gate Bridge, which many thought could not be built because of the continual strong tidal currents. "Now," he said, "the construction men of the West are fully prepared to undertake even bigger things in the post-war world. They look forward to cooperating in harnessing the almost limitless energy of the Pacific basin, which they expect will become the center of the future world."

Both speakers have been closely identified with wartime shipbuilding—Mr. Hilp as a partner in the firm of Barrett and Hilp, which built concrete ships in San Francisco, and Mr. Bechtel as vice-president of the Marinship Corporation at Sausalito, Calif., builder of fast tankers.

Sixth Annual Water Conference to Be Held in October

IT has been announced that the Sixth Annual Water Conference of the Engineers' Society of Western Pennsylvania will be held in the Hotel William Penn, Pittsburgh, Pa., on October 22 and 23. Prominent engineers from all over the country will present papers on a wide variety of timely research problems.

The meeting will take the form of a district meeting in conjunction with a session of the Civil Section of the Engineers Society of Western Pennsylvania.

Hydrologic Data Made Available by C. S. Jarvis

MANY data in the field of hydrology have been collected by C. S. Jarvis, M. Am. Soc. C.E. This material has been published in two pamphlets, entitled "Gleanings from the Field of Hydrology" (reprinted from the 1943 *Transactions* of the American Geophysical Union) and "Supplementary Gleanings" (a 90-page booklet privately published by Dr. Jarvis and now available for general distribution). Included in these two pamphlets are numerous tables of recorded data collected during more than forty years of experience in the field.

A period of more than 120 years of records is covered. Many old records discounted at one time or another have been shown by careful research to be reliable. In some cases all the available and creditable records of rainfall, runoff, evaporation, temperature, river-surface elevations, and related information have been correlated in the tables presented.

Among other data of interest is the first comprehensive hydrograph of the Mississippi River and its principal tributaries, which was compiled by the Mississippi River Commission and released in 1881. Besides records of the Mississippi, there are charts giving information about the Delaware, Gila, Rio Grande, Muskingum, Nile, Concho, Colorado, and Washita rivers.

A limited number of copies of the "Supplementary Gleanings" has been made available by Dr. Jarvis for distribution without charge to members of the Society. A few copies of the earlier "Gleanings" are also to be had, and in addition reprints of his paper, "River Discharge in Brazil," from the 1944 *Transactions* of the American Geophysical Union. All these pamphlets are on file in the Engineering Societies Library.

All inquiries and requests for copies should be addressed to the American Society of Civil Engineers, 33 West 39th Street, New York 18, N.Y.

Planning Neighborhood Shopping Centers

THE use of purchasing power as a yardstick in planning neighborhood shopping sections is urged in the latest study of the National Committee on Housing—"Planning Neighborhood Shopping Centers." The study calls attention to the fact that most communities are over-zoned and over-built as far as neighborhood stores are concerned, and points out that only by advance study and planning can improvements be made in shopping facilities and the physical appearance of communities. Maintenance of real estate values and relief to distressed retail merchants are other problems analyzed in the study, which also outlines methods of rectifying existing conditions and planning new neighborhood shopping areas.

The text of the study is liberally supplemented by tables and other factual data and gives the theoretical area requirements for neighborhood shopping centers together with the number and kinds of stores needed.

The study was made possible through the interest of the Field Foundation, Inc., of New York and Chicago. It sells for $1, and may be obtained from the National Committee on Housing, 512 Fifth Avenue, New York 18, N.Y.

Rehabilitation of Veterans

THE general question of rehabilitation of veterans is considered in a pamphlet, entitled "Rehabilitation: The Man and the Job," recently issued by the National Research Council. Recommendations and suggestions fall into two groups: The one concerned with the discovery and use of the individual's technical skills; the other with methods of securing teamwork and individual cooperation.

The pamphlet constitutes a report of the Subcommittee on Rehabilitation of the Council's Committee on Work in Industry. It may be purchased for 25 cents, upon application to the Publication Office of the National Research Council, 2101 Constitution Avenue, Washington 25, D.C.

Predoctoral Fellowships in the Natural Sciences

THE NATIONAL RESEARCH COUNCIL announces that it is now ready to receive nominations and applications for the predoctoral fellowships in the natural (that is, mathematical, physical, and biological) sciences which it is administering under a grant from the Rockefeller Foundation. These fellowships are intended to assist young men and women, whose graduate study has been prevented or interrupted by the war, to complete their work for the doctorate. It is hoped that these fellowships will do much to accelerate the recovery of the scientific vigor and competence of the country, which is so seriously threatened by the loss of almost two graduate school generations of scientifically trained men and women.

The annual stipend will be $1,200 for single persons and $1,800 for married men. In general it is expected that each recipient will spend at least eleven months per year on academic work. An additional allowance, up to $500 per year, will be made for tuition fees. Fellowships granted to individuals who are eligible for educational support from the "G.I. Bill of Rights" will be at such stipends as to bring the total income from the two sources up to the above rates.

Each fellow, before entering on his graduate studies, will submit for review by the Committee on Predoctoral Fellowships a schedule, approved by the dean of his graduate school, for the completion of his work for the doctorate. This schedule, as approved by the committee, will constitute an informal agreement upon the basis of which stipend payments will be made.

At the discretion of the university concerned, the fellowship stipend may be supplemented by university grants. All such supplementary sources of income should be made a matter of record with the committee. The progress of the fellows will be subject to periodic review by the committee, which reserves the right to cancel fellowships when in its judgment satisfactory progress is not being maintained.

Prospective candidates for these fellowships are urged to apply at once even though they may be unable to undertake their graduate study in the immediate future. Information concerning these fellowships and nomination-application blanks are being mailed out widely to graduate schools and wartime research laboratories. They may also be obtained by writing directly to the Secretary, Committee on Predoctoral Fellowships, National Research Council, 2101 Constitution Avenue N.W., Washington 25, D.C.

NEWS OF ENGINEERS

Personal Items About Society Members

GEORGE W. CASE, director of the Engineering, Science and Management War Training Program of the U.S. Office of Education, retired as dean of the college of technology of the University of New Hampshire on June 30 of this year. Dean Case has been on leave of absence from the University since November 1940 in order to participate in the administration of the college-level training programs financed by the federal government.

CHARLES T. DICKEMAN was recently promoted from the rank of captain in the Civil Engineer Corps of the U.S. Navy to that of commodore. He has been in command of the Fifth Naval Construction Brigade on Guam since June.

R. C. VOGT and W. J. IVERS announce the formation of an engineering partnership under the name of Vogt, Ivers, and Associates, with offices at 710 Second National Bank Building, Cincinnati 2, Ohio. Mr. Vogt was formerly principal engineer in charge of the engineering division of the U.S. Engineer Office in Cincinnati, and Mr. Ivers was director of structural engineering for A. M. Kinney, Inc., of the same city.

W. C. POLKINGHORNE is now full professor and head of the department of civil engineering at Michigan College of Mining and Technology, having been advanced from the rank of associate professor.

ADOLPH J. ACKERMAN has resigned as director of engineering for the Dravo Corporation at Pittsburgh, Pa., in order to engage in hydroelectric development work in South America. Although in recent years Mr. Ackerman has been engaged in heavy construction work and shipbuilding, the present step marks his return to the field of water power engineering, in which he has had wide experience.

RUSSELL L. KLOTZ, until lately engineer for the Special Engineering Division of the Panama Canal, has been appointed chief of the Division. Mr. Klotz has been employed on the Isthmus since 1929—for the past six years on the Third Locks Project.

OLE SINGSTAD announces that he is retiring, effective October 1, from his position as chief engineer of the New York City Tunnel Authority. He will continue his consulting practice with offices at 14 Whitehall Street, New York 4, N.Y.

THOMAS A. BERRIGAN, recently released from active duty with the Civil Engineer Corps of the U.S. Naval Reserve, has been appointed director and chief engineer of the Sewerage Division of the Metropolitan District Commission, Boston, Mass. He is also serving as associate commissioner on the Metropolitan Water Supply Commission.

HAROLD S. CARTER, formerly professor of civil engineering at Utah State Agricultural College, has accepted a position in a similar capacity at the University of Utah. Professor Carter is president of the Intermountain Section of the Society.

REVOE C. BRIGGS, principal assistant to the district engineer for the U.S. Geological Survey in California, has left San Francisco for a year's detail to Caracas, Venezuela, as adviser in hydrology. He is to organize a national program of surface-water investigations, to be continued by Venezuelan engineers following his return to the United States. Arrangements for the visit were made by the Department of State at the request of the Venezuelan government.

A. S. MACGREGOR has been promoted from the rank of first lieutenant in the U.S. Army to that of captain. Captain Macgregor, who is assistant post engineer at Fort Wadsworth, N.Y., was building construction engineer with the New York State Department of Public Works before being called to active duty.

ROBERT A. MONROE has been appointed chief design engineer of the Tennessee Valley Authority, succeeding GEORGE R. RICH who has resigned to enter private practice. Mr. Monroe has been with the TVA since 1937—most recently in the capacity of assistant chief engineer. Mr. Rich will be associated with Charles T. Main, Inc., in Boston, Mass.

HORACE P. HINCKLEY is now general manager of the Bear Valley (California) Mutual Water Company, principal supplier of surface water in the San Bernardino Valley. Mr. Hinckley was previously an engineer for the San Bernardino Valley Water Conservation District.

THOMAS J. FRATAR was recently promoted from the rank of second lieutenant in the U.S. Army to that of first lieutenant. He has served for over two years with the Persian Gulf Command in Iran and, at present, is a valuation engineer for the Third Military Railway Service there.

JAMES J. WALSH, San Francisco consultant, has been appointed a member of the San Francisco City Planning Commission.

DAVID MCCOACH, Jr., major general, Corps of Engineers, U.S. Army, has had the Oak Leaf Cluster added to his Distinguished Service Medal for "assuming duties of great responsibility soon after the invasion of Southern France" in his capacity as chief engineer, Allied Force Headquarters, Mediterranean Theater of Operations.

LEON G. WILLIAMS, formerly resident representative for the Chicago consulting firm of Greeley and Hansen at Norfolk, Va., has accepted a position with Jones and Henry, consulting hydraulic and sanitary engineers, of Toledo, Ohio.

PHILIP HARRINGTON has resigned as Commissioner of Subways and Superhighways for the City of Chicago in order to become chairman of a recently appointed seven-man Chicago Transit Board, which will direct operations of the Chicago Transit Authority.

NEAL B. GARVER, since 1921 bridge engineer for the Arkansas State Highway Commission, has been promoted to the position of chief engineer.

NED D. BAKER, previously office engineer for C. C. Kennedy, of Benicia, Calif., has accepted a position on the engineering staff of the East Bay (California) Municipal Utility District.

HERBERT P. NILMEIER has severed his connection with the engineering staff of the University of California, where he was an instructor, in order to become a sanitary engineer with the 12th Naval District.

ARTHUR L. PAULS, until lately chief construction engineer for the Tennessee Valley Authority at Knoxville, Tenn., has become associated with the Morris-Knudsen Company, with headquarters in San Francisco. Other members of the Society who have recently left the staff of the TVA are G. L. WILLIAMS, now with the James Leffel Company at Springfield, Ohio, and PORTLAND P. FOX, who has accepted a position with the U. S. Bureau of Reclamation in Denver, Colo.

EDWARD T. FOSTER, colonel, Field Artillery, U.S. Army, was recently awarded the Legion of Merit for "exceptionally meritorious conduct in the performance of outstanding services" during the Italian campaign. Serving as assistant chief of staff, G4, of the base at Leghorn, Italy, Colonel Foster was in charge of the plans for equipping the major part of the land forces to invade southern France from Italy. Before entering the Army, Colonel Foster was superintendent of the construction department for the Omaha (Nebr.) Steel Works.

J. O. OSTERBERG has been appointed assistant professor of civil engineering at the Technological Institute, Northwestern University. His time will be divided between teaching and research for the Society's Committee on Sampling and Testing of the Soil Mechanics and Foundations Division, for which he has been research engineer since 1943.

JOHN R. OSBORN, formerly instructor in civil engineering at the Agricultural and Mechanical College of Texas, has joined the staff of the American University of Beirut, Beirut, Lebanese Republic.

LEONARD C. URQUHART has received a temporary promotion from the rank of lieutenant colonel in the Corps of Engineers, U.S. Army, to that of colonel. He is chief of the War Construction Section, with headquarters in Washington, D.C. Colonel Urquhart's home is in Ithaca, N.Y.

JAMES WENDELL ADAMS was recently appointed chief engineer of the New York firm of McLaughlin-Carr Associates in charge of its airport development work. He was previously engineer in charge of airport engineering, design, construction, and maintenance for the Curtiss-Wright Corporation, with which he became connected in 1929.

MARVIN G. STURGEON, who is in charge of the operations unit in the flight operations section of the Army Air Force Training Command headquarters at Fort Worth, Tex., has been promoted from the rank of major to that of lieutenant colonel. Prior to entering the service in 1940, Colonel Sturgeon was in the U.S. Engineer Office at Portland, Ore.

CLIFFORD D. WILLIAMS, formerly professor of structural engineering at Fenn College, has assumed new duties as acting head of the department of civil engineering at the University of Florida. He has been on leave of absence from Fenn College doing war work, his most recent assignment having been that of assistant supervising engineer for the J. E. Greiner Company, of Baltimore, Md.

MAXWELTON S. CAMPBELL is now connected with the H. D. Fowler Company, a water-works supply firm in Seattle, Wash. Until recently he was chief of the Division of Public Health Engineering of the Washington State Department of Health.

GEORGE M. MARCH, until lately associate engineer for the Kansas State Board of Health, has accepted the position of city manager of Independence, Kans.

[*Editor's Note:* In the August issue it was erroneously stated that James M. Montgomery, M. Am. Soc. C.E., was manager of the Defense Plant Corporation's magnesium plant at Las Vegas, Nev. The Los Angeles firm of J. M. Montgomery and Company, with which Mr. Montgomery has no connection, are managers of the project for the Reconstruction Finance Corporation.]

DECEASED

HENRY GIRDLESTONE ACRES (M. '16) president of H. G. Acres and Company, Ltd., of Niagara Falls, Canada, died in a hospital in Toronto on September 4, 1945. His age was 65. In 1905 Mr. Acres placed in operation the first 10,000-hp turbine ever built, and in 1907 constructed the first 10,000-volt transmission line. He was for many years hydraulic engineer for the Hydroelectric Power Commission of Ontario, and in 1930 became president of H. G. Acres and Company, Ltd. One of Canada's leading hydroelectric engineers, Mr. Acres was especially known for his work in designing and supervising the Chippawa-Queenston development of the Hydroelectric Power Commission of Ontario and the Shipshaw development of the Aluminum Company of Canada at Arvida, Quebec.

GEORGE BERRY (M. '05) retired civil engineer of Brooklyn, N.Y., died at his summer home in Brooklyn, N.Y., on August 2, 1945. Mr. Berry was for many years assistant engineer in the Brooklyn Bureau of Highways. He retired in 1936.

WILL HAZEN BOUGHTON (Assoc. M. '07) retired engineer and accountant, died at Olean, N.Y., on July 27, 1945, at the age of 78. Mr. Boughton spent his earlier career teaching—from 1894 to 1903 he was professor of civil engineering at Denison University, and from 1903 to 1911 at West Virginia University. From 1911 to 1915 he was treasurer and business manager of Vassar College, and from 1917 to 1937 treasurer of Smith Brothers, Inc., of Poughkeepsie, N.Y. He then became comptroller of Smith Brothers, serving until his retirement two years ago.

WALDO EMERSON BUCK (M. '89) of Worcester, Mass., died several months ago. Mr. Buck, who was 89, had spent much of his career as president and treasurer of the Worcester Manufacturers Mutual Insurance Company. Earlier he had been engaged in railroad work, and for some years he was agent and local manager for the Winnepissiogee Water Power Company of Lake Village, N.H.

JAMES FRANCIS CULLEN (M. '06), retired engineer of construction for the Pennsylvania Railroad, Philadelphia, Pa., died on February 24, 1945. He was 80. Mr. Cullen spent his entire career with the Pennsylvania Railroad, with which he became connected in 1890. He was construction engineer for a number of years prior to his retirement in 1937.

WILLIAM ROBERT DUNHAM, JR. (M. '06) retired engineer of Providence, R.I., died in North Haven, Conn., on August 13, 1945. Mr. Dunham, who was 74, devoted his career to street and inter-urban electric traction. Serving first with the Union Railway of Providence, he then became division engineer and, later, engineer of maintenance of way for the Connecticut Company. From New Haven he went to Detroit, where he served the Detroit Municipal Railway as executive engineer and research expert.

PAUL EMERY (M. '39) project chief engineer for the McLean Contracting Company at Norfolk, Va., died suddenly on August 27, 1945, at the age of 54. From 1912 to 1917 Mr. Emery was with the Topeka (Kans.) Bridge and Iron Company; from 1917 to 1918, with the Lackawanna Bridge Company at Buffalo, N.Y.; from 1919 to 1925, with the American Rio Grande Land and Irrigation Company at Mercedes, Tex.; from 1927 to 1928, with the American Bridge Company at Ambridge, Pa.; and from 1929 to 1940, office engineer for the Kansas City Bridge Company at Kansas City, Mo. For the past five years he had been engineer in charge of construction of the work performed by the McLean Contracting Company at the Norfolk Naval Base.

OLAF LAURGAARD (M. '14) resident plant engineer for the Bethlehem Alameda (California) Shipyard during the war, died in San Francisco, Calif., on June 23, 1945. His age was 65. Mr. Laurgaard was project engineer for the state of Oregon from 1913 to 1915; city engineer of Portland, Ore., from 1917 to 1934; engineer with the U.S. Bureau of Reclamation in Denver, Colo., from 1934 to 1936; and construction engineer for the TVA on Hiwasee Dam from 1937 to 1940. In the latter year Mr. Laurgaard established a consulting practice in Portland, and since 1942 he had been resident plant engineer at the Bethlehem Alameda Shipyard.

ENGBERT A. LEE (M. '20) who was with the Climax Molybdenum Company, of Golden, Colo., died on June 7, 1945, at the age of 75. Beginning in 1905, Mr. Lee was for a number of years connected with the American Smelting and Refining Company—for part of this period as chief engineer of the Colorado department. Later he was with the U. S. Zinc Company at Amarillo, Tex., and from 1931 to 1941 he maintained a consulting practice in Denver, Colo.

DONALD GRANT MCKIM (Assoc. M. '29) senior resident engineer for the Texas State Highway Department, Houston, Tex., died on July 29, 1945. Mr. McKim, who was 58, had been in the Texas State Highway Department for over twenty years. During this period he had been resident engineer at Austin, New Braunfels, and Livingston, Tex.

JOHN HENRY NEESON (M. '30) Director of Public Works, Philadelphia, Pa., died on September 1, 1945, at the age of 61. Mr. Neeson, who was past-president of the American Society of Municipal Engineers, had been in the Philadelphia municipal service since 1906, when he became assistant superintendent of bridges. His activities included reorganization of the city's street and ash-collection services, city planning, development of the drainage and sewage disposal systems, and direction of the city's WPA program. From 1928 until 1940 Mr. Neeson was chief of the Bureau of Engineering Surveys and Zoning, and in 1940 he was appointed director of public works. In 1943 he was made vice-chairman of the new City Planning Commission.

SHEPARD BROWN PALMER (M. '11) consulting engineer of Norwich, Conn., died on August 17, 1945, at the age of 74. In 1894 Mr. Palmer became associated with Charles E. Palmer, and in the engineering firm of Chandler and Palmer he had served for a long period as city engineer of Norwich. Long a member of the State Board of Civil Engineers and, later, of the Board for the Supervision of Dams and Reservoirs, he was closely identified with the development of a number of water works in eastern Connecticut.

ROBERT CHARLES RATCLIFFE (Assoc. M. '36) who was on the staff of the U.S.

Bureau of Reclamation at Laramie, Wyo., died in a hospital in Cheyenne, Wyo., on August 16, 1945. He was 42. Mr. Ratcliffe had been with the Iowa State Highway Commission and the Colorado State Highway Department, and was for some years chief of the Department of Public Works for Grand Junction, Colo. More recently he had been associate engineer in the U.S. Engineer Office at Albuquerque, N.Mex., and city engineer of Laramie, Wyo.

WALTER LEWIS STISULIS (Jun. '41) lieutenant, U.S. Navy, died recently. Lieutenant Stisulis was 28, and an alumnus of the University of Maine, class of 1941. Before joining the Navy, he was with the Maine State Highway Commission at Augusta. His home was in Mexico, Me.

FRANK HERBERT TODD (M. '89) engineer for City Water Works, El Paso, Tex., died there on October 19, 1944, at the age of 85. Mr. Todd devoted the greater part of his engineering career to the field of water supply, his specialty being underground development. For a number of years, also, he maintained a general consulting practice in El Paso and, later, in San Juan, Puerto Rico. At one time he was city engineer of El Paso.

ALBION NOYES VAN VLECK (M. '28) consulting engineer of New York, N.Y., died in a hospital there on August 22, 1945, at the age of 59. Starting his career in 1908, Mr. Van Vleck served as assistant chief engineer for Guy Lowell, New York and Boston architect, from 1913 to 1917. In the latter year he entered the Army and saw service in the artillery, rising to the rank of major. After the war he became assistant general manager for the Saginaw (Mich.) Shipbuilding Company, and from 1922 to 1927 was chief engineer for J. E. R. Carpenter, New York architect. In 1927 he became a partner in the consulting firm of Hurlbut and Van Vleck, and in 1938 was made first deputy housing commissioner, resigning two years later to resume private practice. Long active in Masonic affairs, Mr. Van Vleck was executive secretary of the Military and Naval Services Committee of the Grand Lodge of Masons, New York.

LIONEL RENE VITERBO (M. '18) consulting engineer of St. Louis, Mo., died on August 16, 1945. He was 66. Mr. Viterbo was born in Constantinople, Turkey, and educated in France, coming to the United States in 1902. From 1905 on he was a member of the St. Louis and Chicago consulting firm of Brussel and Viterbo, in charge of the building of various notable structures in Chicago and the Southwest. Coincidentally, for most of the period he was also, successively, vice-president and president of the Reinforced Concrete Company of St. Louis.

CHARLES BENJAMIN WING (M. '08) emeritus professor of structural engineering at Stanford University, Palo Alto, Calif., died in that city on August 22, 1945. His age was 81. Early in his career Professor Wing taught at Cornell and the University of Wisconsin. In 1892 he became professor of structural engineering at Stanford University, and remained there until his retirement in 1929 with the title of emeritus professor. Coincidentally, for most of this period, he maintained a consulting practice in Palo Alto. During the first World War Professor Wing served overseas with the 23d Engineers, having the rank of lieutenant colonel.

[*Editor's Note:* It is greatly regretted that, through an error, the death of Emilio Guaroa Joubert, Assoc. M., was reported in the August issue. The Society received official notice of Mr. Joubert's death through a mistake identifying him with his father of the same name, who died recently. At present Mr. Joubert is living in Monte Cristy, Dominican Republic.]

Changes in Membership Grades

Additions, Transfers, Reinstatements, and Resignations

From August 10 to September 10, 1945, Inclusive

ADDITIONS TO MEMBERSHIP

AMES, GORDON MILLER, JR. (Assoc. M. '45), Structural Engr., Magnolia Petroleum Co., Box 900 (Res., 1315 Green Drive), Dallas 8, Tex.

ANDERSON, ENOCH WILLIAM (Jun. '45), Airways Engr., Civ. Aeronautics Administration, 84 Marietta St., N.W., Atlanta (Res., 112 North Madison St., College Park), Ga.

ANDREWS, GEORGE HOWARD (Jun. '45), Engr., State Dept. of Highways, Transportation Bldg. (Res., 211 Maple Park), Olympia, Wash.

BAKER, HAROLD TOWER (M. '45), Project Engr., The H. K. Ferguson Co., Hanna Bldg. (Res., 3670 Latimore Rd.), Cleveland 22, Ohio.

BANG, SVEND AAGE (Jun. '45), Lt. (jg), U.S.N.; Keewatin, Minn.

BARNES, ARTHUR RUSSELL, JR. (Assoc. M. '45), Designer, J. R. Worcester & Co., 79 Milk St. Boston (Res., 37 Willow St., Reading), Mass.

BAYSINGER, RALPH CLAYTON (M. '45), Asst. Engr., Union Pacific R.R., 851 Pittock Block, Portland 5, Ore.

BELLOWS, WARREN SYLVANUS, JR. (Assoc. M. '45), Lt., CEC, U.S.N., 10th Naval Constr. Brigade, Care, Fleet Post Office San Francisco, Calif.

BENSON, JOHN BURT, JR. (Jun. '45), Civ. Engr., J. B. Converse & Co., Inc., Box 1084, Mobile, Ala.

BISHOP, ARTHUR DALE (M. '45), Bridge Engr., State Dept. of Highways (Res., 375 Elm St.), Montpelier, Vt.

BLAKE, EDWARD FRANKLIN (M. '45), Chf. Engr., McLean Cont. Co., 1301 Fidelity Bldg. (Res., 109 Overhill Rd.), Baltimore 10, Md.

BLUESTEIN, ED. (M. '45), Dist. Engr., State Highway Dept., Atlanta, Tex.

BOND, AUBREY HOODENPYL (M. '45), Col., Corps of Engrs., U.S. Army, Antilles Div. Engr., Army Post Office 851, Care, Postmaster, Miami, Fla.

BRODIE, OMER HAROLD (Jun. '45), Carpenter's Mate 1/C, U.S.N., Constr. Battalion Maintenance Unit 513, Care, Fleet Post Office, San Francisco, Calif.

BROWN, CHESTER IRA (Assoc. M. '45), Associate Highway Engr., State Dept. of Public Works, 1657 Riverside (Res., 2180 Terrace St.), Redding, Calif.

BROWN, REUBEN FRANCIS (Assoc. M. '45), Supt. of Sewer Maintenance, Dept. of Public Works, Bureau of Maintenance & Sanitation, Room 746, City Hall, Los Angeles 12, Calif.

BRYNER, CLIFFORD GAULT (Jun. '45), Ensign, CEC, U.S.N.R.; 4150 South State St., Murray 7, Utah.

BURGER, CLARENCE CHEW, JR. (M. '45), Head Engr., Chf., River and Harbor Div., Office, Chf. of Engrs., Room 6119, New War Bldg., Washington, D.C. (Res., Burnt Mills Hills, Silver Spring, Md.)

CALLAWAY, THOMAS CHAUDION (Assoc. M. '45), Res. Engr. on Buildings, Robert and Co., Inc. (Res., 2306 Blvd. Granada, S.W.), Atlanta, Ga.

CAPES, CHARLES FREDERICK (M. '45), Prin. Highway Engr., P-6, U.S. Public Roads Administration, Custom House, Denver, Colo.

CHEEK, FRANK JACOBS, JR. (M. '45), Prof., San. Eng., Univ. of Kentucky, Lexington 29, Ky.

CLOUGH, RICHARD HUDSON (Jun. '45), Lt. (jg), U.S.N.; 924 North 11th St., Albuquerque, N. Mex.

COLDITZ, HUGO (Assoc. M. '45), Civ. Engr., TVA, 320 Union Bldg., Knoxville (Res., 316 Knox Ave., Fountain City 18), Tenn.

COOK, ALPHA LANOY (Assoc. M. '45), Associate Highway Engr., U.S. Public Roads Administration, Census Bldg., Suitland, Md. (Res., 1218 Forty-Fourth Pl., S.E., Washington, D.C.)

COYKENDALL, KENNETH PHILIP (M. '45), Lt. Comdr., CEC, U.S.N.R., Asst. Public Works Officer, Naval Air Station, Pensacola, Fla. (Res., Georges Mills, N.H.)

CUSHMAN, TOWNSEND HAMILTON, JR. (Jun. '45), Ensign, CEC, U.S.N.R., 302d Naval Constr. Battalion, Care, Fleet Post Office, San Francisco, Calif.

CUSICK, PATRICK JOSEPH, JR. (Jun. '45), Lt., CEC, U.S.N.R., Care, Officer-in-Chg. of Constr., Bureau of Yards and Docks Contracts, Army Post Office 605, Care, Postmaster, Miami, Fla.

DADDISMAN, REX ALLEN (M. '45), Lt. Col., Corps of Engrs., U.S. Army, AAF Liaison Officer, 730 Pittock Block, Portland 5, Ore.

ECKHOFF, HAROLD EUGENE (Jun. '45), Ensign, CEC, U.S.N.; Harvel, Ill.

ENGEL, ROBERT FRANCIS (Assoc. M. '45), Lt., U.S.N., Hull Constr. Officer, Supervisor of Shipbuilding, Consolidated Steel Corp., Orange, Tex.

ESCAMILLA, JUAN FRANCISCO (Jun. '45), Civ. Engr. (Army Constructors), Dept. of War, Escuela Politecnica (Res., 13 Calle Poniente No. 23), Guatemala, Guatemala.

FAIRFIELD, GUY MOORE (Assoc. M. '45), Constr. Engr., Federal Works Agency, Box 132, Alton, Ill.

FULLERTON, JOHN HUBBARD (Jun. '45), Ensign, U.S.N.R.; 14 Rawston Rd., Roslindale 31, Mass.

GILDEA, ROBERT EDWARD LEE (Assoc. M. '45), Associate Prof., Civ. Eng., Virginia Polytechnic Inst., Blacksburg, Va.

GILLIARD, WILLIAM HENRY (Assoc. M. '45), Engr., Chf., Civ. Eng. Section, Ordnance Research and Development Center, Aberdeen Proving Ground (Res., 43 Defense Drive), Aberdeen, Md.

GOODIER, ROBERT DOUGLAS (Jun. '45), Flight Test Analyst, North Am. Aviation, Inc. (Res., 10930 Buford Ave.), Inglewood, Calif.

GRIBBLE, JAMES MCCONVILLE (Jun. '45), Ensign, CEC, U.S.N.R., 72d Naval Constr. Battalion, Care, Fleet Post Office, San Francisco, Calif.

HERRMANN, WILLIAM ANTHONY (Jun. '45), Ensign, CEC, U.S.N.; 1262 Pennsylvania Ave., St. Louis, Mo.